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Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

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Papers

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Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

PORE PRESSURE IN CONCRETE DAMS

Chong-Hung Zee[1]
(Proc. Paper 1597)

SYNOPSIS

In this paper a detailed discussion on the mechanics of uplift force acting within a concrete body is presented first by conceiving the concrete as a "space frame" structural body; thus the idea of the effective uplift area is explained. Furthermore, as water percolates through the "space frame" structural body, the seepage force occurs. The resultant of uplift and seepage forces is the pore pressure force, which may be deduced from the potential theory on flow through a pervious medium. Following this idea the water dammed up by a concrete dam does not act entirely on the upstream face of the dam, it acts successively within the dam; accordingly the stresses within the dam are changed. Applying the conception of seepage face height in well hydraulics, the intensity of pore pressure at the drainage system near the upstream face of the dam as well as the most effective spacing of vertical drains can be determined in laboratory by means of electrical analogy or other comparable analysis.

INTRODUCTION

A review of literature about uplift pressure on concrete dams shows that two fundamental questions are in involved: one is regarding the effective uplift area; the other, the intensity of the uplift pressure acting on the effective area. Various authorities(1,2,3) have presented experimental evidence showing that the effective area is 85%-100%; theoretical deductions(4) based on cutting imaginary section give a value of 100%. Discussion on the intensity of the uplift pressure supported by practical measurements has been limited

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more or less near the base of concrete dams.(5)

Concrete, known as a pervious material, is subject to the action of seepage force when water flows slowly through it. In the case of concrete dams, both the uplift and the seepage forces exist. By combining the uplift and the seepage forces, the pore pressure force is introduced. Based on the fundamental principles of mechanics, a thorough discussion of pore pressure acting within a concrete dam will be presented in this paper. It is hoped that this investigation may give the practical designers some idea in dealing with the pore pressure within concrete dams.

Concrete Visualized as a "Space Frame" Structural Body

It is a well-known fact that concrete has about 12-15% voids and neat cement has approximately 28-48% voids.(6) In the practical case the sand and cement fill the voids of coarse aggregates, and the cement fills the voids of sand. The interaction among the coarse aggregates, sand, and cement resembles a "space frame" structural body: the coarse aggregates and sand act as the "large solid part," and the cement acts as the "connecting members" in the "space frame" structural body. From the stress point of view, the contact surface between cement and sand particles or coarse aggregates should be a surface, not a point, otherwise it is impossible to explain the local stress condition within the concrete body when it is under tension or compression. Unfortunately, in ordinary material testing, the idea of contact stress within a concrete block has not been introduced. In order to bring out the importance of this contact stress, Prof. Terzaghi mentioned that the tensile contact strength of concrete is greater than 10,000 lbs. per sq. in.(6) This high tensile strength illustrates how the conception of strength in "space frame" differs from the conception of the tensile strength of concrete in the ordinary sense.

Based on the conception of contact stress, the surface of failure is a surface along which that some of the "connecting members" cannot resist the shearing stress and some of them cannot resist the normal stress. However, the location of the surface of failure is governed by "space frame" structural condition, which constitutes one of the physical properties of the material. Actually the material is not homogeneous, that is, the material does not have the same form of "space frame" structure at every point in the body. Consequently, the resistence of the material is not always the same. This idea can be illustrated by an example in structural engineering. If two trusses of the same number of members and the resistence of every member of these two trusses is the same, but these two trusses have different structural form, the resisting load of these two trusses is not the same. It is to be emphasized that the "connecting member" strength does not vary with the form of truss or the form of "space frame" structure. The previous discussion may explain why concrete strength testing data are scattered even the specimen are treated in the same way.

Stresses Within a "Space Frame" Submerged in Static Water

Suppose that a "space frame" being under external loads is submerged into static water and the stresses in the "space frame" are analyzed, then four kinds of force may be classified as to produce contact stresses on a

section (or sections) of the "space frame":

1) External force (E) known as a resultant from external loads;

2) Weight (W) being the weight of the part of the "space frame" above an investigated section;

3) Bouyant force (B) referring to the weight of water displaced by the "space frame" above the investigated section; and

4) Water pressure force (P) resulting from the fact that there is no upward water pressure on every small section (see fig. 1) to balance the corresponding downward pressure.

On every small section the stresses are divided into three elements:

1) Moment $M = M_e + M_w + M_b + M_p$ where M_e is the moment due to E etc.;

2) Shear $S = S_e + S_w + S_b + S_p$ where S_e is the shear due to E etc., and S_p is equal to zero because the water pressure is always perpendicular to the small investigated section;

3) Normal force (tension or compression) $N = N_e + N_w + N_b + N_p$ in which N_e is the normal force due to E etc.;

and each element consists of four factors as shown in the above equations.

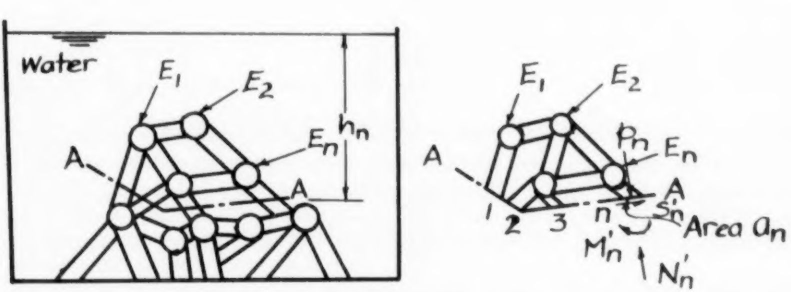
Considering the section A-A in fig. 1:

In Air:		In Water:	
$M_i = M_{ei} + M_{wi}$	$(1)_i$	$M_i' = M_{ei} + M_{wi} + M_{bi} + M_{pi}$	$(1)_i'$
$S_i = S_{ei} + S_{wi}$	$(2)_i$	$S_i' = S_{ei} + S_{wi} + S_{bi} + S_{pi}$	$(2)_i'$
$N_i = N_{ei} + N_{wi}$	$(3)_i$	$N_i' = N_{ei} + N_{wi} + N_{bi} + N_{pi}$	$(3)_i'$

$(i = 1,2,----,n)$

Let r = unit weight of water;

σ = unit weight of solid material,



"SPACE FRAME" STRUCTURAL BODY

FIG. 1.

then in the above equations:

$$M_i' = M_{ei} + M_{wi} + M_{bi} + M_{pi} = M_{ei} + \frac{\gamma_s - \gamma}{\gamma_s} M_{wi} + M_{pi} \qquad (1)_i''$$

$$S_i' = S_{ei} + S_{wi} + S_{bi} \qquad = S_{ei} + \frac{\gamma_s - \gamma}{\gamma_s} S_{wi} \qquad (2)_i''$$

$$N_i' = N_{ei} + N_{wi} + N_{bi} + N_{pi} = N_{ei} + \frac{\gamma_s - \gamma}{\gamma_s} N_{wi} + \gamma h_i a_i \qquad (3)_i''$$

$$(i = 1,2,----,n)$$

where a_i = sectional area of the individual member;

h_i = the distance from the center gravity of the section a_i to the free water surface;

$N_{pi} = \gamma h_i a_i$ acting perpendicularly to the section a_i; and

$M_{pi} = \gamma h_i a_i$ x the distance between center of pressure and center of gravity of the area a_i.

On comparing equations:

$(1)_i$ and $(1)_i''$, the moment reduces $\frac{\gamma}{\gamma_s} M_{wi}$, which is entirely due to bouyant effect, and the moment is increased by the amount of M_{pi} beyond the general reduction because section A-A is irregular and not horizontal;

$(2)_i$ and $(2)_i''$, the shear reduces $\frac{\gamma}{\gamma_s} S_{wi}$, which is due to bouyant effect;

$(3)_i''$ and $(3)_i''$, the normal force reduces $\frac{\gamma}{\gamma_s} N_{wi}$ and increases $\gamma h_i a_i$ due to water pressure on a_i.

A conclusion drawn from the above investigation follows:

1) the existence of water reduces generally the stress elements of moment, shear, and normal force an amount of $\frac{\gamma}{\gamma_s}$ of that of the corresponding element in air because of bouyancy;

2) beyond the general reduction, the moment is increased by the amount of M_{pi}, and the normal force is increased by the amount of $\gamma h_i a_i$.

Suppose that the section A-A in fig. 1 is horizontal, by principles of statics,

$$\sum_{i=1}^{n} \frac{\gamma_s - \gamma}{\gamma_s} S_{wi} = 0$$

and $M_{pi} = 0$ because all a_i's are horizontal. Furthermore, macroscopically, the moments on all a_i's are of negligible importance; hence the moment equations may be dropped. The ratio of the sum of a_i's across the section A-A to the gross area of that section for concrete has been well presented elsewhere(1,2,3); it is known as varying from $0\pm\%$ to 15%, but the lower limit may be interpreted as an experimental error because $\sum_{i=1}^{n} a_i$ for any section is not zero.

Pore Pressures Acting Within Concrete Dams

If the "space frame" is submerged into water which percolates through the "space frame," then the stresses in the "space frame" are somewhat different from the previous analysis; besides the foregoing mentioned four kinds of force, an additional force resulting from percolation is introduced. Macroscopically, in dealing with water the "space frame" is a pervious medium; microscopically, cavities in the "space frame" form a series of curved pipes in the direction of streamlines when water percolates through it. The law governing flow through a pervious medium is known as Darcy's law, which may be expressed as

$$v = -k\frac{\partial h}{\partial n_v} \tag{4}$$

v = velocity;
k = permeability;
h = peizometric head;
n_v = distance measured in the direction of v or streamline;
$\partial h/\partial n_v$ = peizometric head gradient in the direction of n_v.

Needless to explain that the term $\partial h/\partial n_v$ is the resulting effect of the resistence of those curved pipes existing within the pervious medium to the flowing fluid. Once the geometric boundary configuration of the pervious medium is fixed, there is only one flow pattern within that geometric configuration and it is independent of the permeability, k, of the pervious medium so far as the Darcy's law is held true. Therefore, the force acting on the pervious medium due to the flowing water is measured by $\partial h/\partial n_v$. There is another force acting on the pervious medium known as bouyant force which results from the elevation gradient, $\partial z/\partial n_z$, due to the weight of water. Of this gradient, n_z is measured in the vertical direction and z is the elevation head.

As will be proved, the resultant of $\partial h/\partial n_v$ and $\partial z/\partial n_z$ is $\partial(p/\gamma)/\partial n_p$, and p is designated here as pore pressure, n_p is in the direction perpendicular to the constant p curve: written in vector form,

$$(\overrightarrow{\partial z/\partial n_z}) + (\overrightarrow{\partial h/\partial n_v}) = (-\overrightarrow{\partial (p/\gamma)}/\partial n_p) \tag{5}$$

The well-known Bernoulli's equation in fluid mechanics is

$$H = z + (p/\gamma) + (v^2/2g) \tag{6}$$

where H = total head at a particular point;

z = elevation at that particular point above a certain datum;

p/γ = pressure head at that particular point;

$v^2/2g$ = velocity head at that particular point.

When v is small in flow through pervious medium, $v^2/2g$ is very small and may be neglected; therefore

$$H = z + (p/\gamma) = h \text{ (piezometric head)} \tag{7}$$

Following operational rules in vector analysis,

$$\text{grad.}\,(h) = \text{grad.}\,(z + (p/\gamma))$$
$$= \text{grad.}\,(z) + \text{grad.}\,(p/\gamma) \qquad (8)$$

Therefore,

$$(\overrightarrow{\partial h/\partial n_v}) = (\overrightarrow{\partial z/\partial n_z}) + (\overrightarrow{\partial(p/\gamma)/\partial n_p})$$
$$\text{or} \quad (\overrightarrow{\partial z/\partial n_z}) + (\overrightarrow{-\partial h/\partial n_v}) = (\overrightarrow{-\partial(p/\gamma)/\partial n_p}) \qquad (5)$$
$$\text{or} \quad (\overrightarrow{\gamma\partial z/\partial n_z}) + (\overrightarrow{-\gamma\partial h/\partial n_v}) = (\overrightarrow{-\partial p/\partial n_p}) \qquad (5a)$$

From the point of view of stress analysis, the factor, $-(\partial p/\partial n_p)$, is much more important than the other two and it measures the resulting effect of bouyant force of water within the pervious medium and the force acting on the pervious medium applied by the flowing water (see fig. 2).

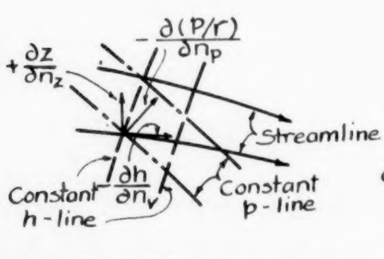
The factor $(-\gamma\partial h/\partial n_v)$ is known as seepage force in soil mechanics.

Considering the simplest case of a triangular section of a pervious body damming up water behind it as shown in fig. 3, the streamlines, lines of equi-pressure, lines of equi-elevation and lines of equi-peizometric head are all shown in the figure. Applying the seepage force idea in soil mechanics, the seepage force acting within a unit length of the pervious body is

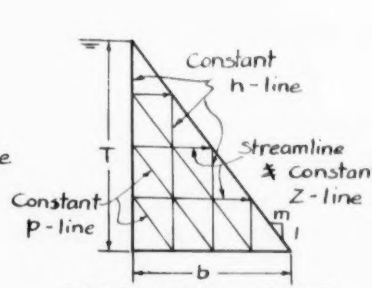
$$\gamma(\partial h/\partial n_v) \times \text{(volume of the pervious body) (horizontal direction)}$$
$$= (\gamma T/b) \times (Tb/2) \qquad (9)$$

Physically, this force, though has the magnitude of $\gamma T^2/2$ which is the total water pressure as if the above triangular body were impervious, is the sum of all small forces acting within the pervious body as shown in fig. 4; they act successively within the pervious body.

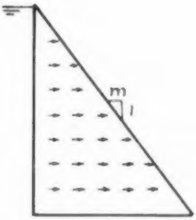
The other force, the bouyant force, is expressed as $\gamma\partial z/\partial n_z$. It acts vertically and also successively within the pervious body as shown in fig. 5. When any horizontal section is taken for the investigation of stresses of the

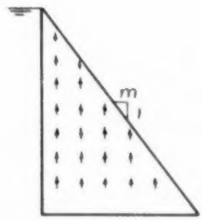


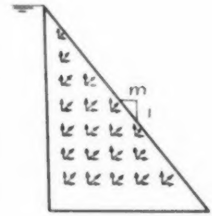
VARIOUS POTENTIAL COMPONENTS IN THE FIELD OF FlOW IN PERVIOUS MEDIUM
FIG. 2.



POTENTIAL FIELD IN A TRIANGULAR SECTION PERVIOUS MEDIUM BODY
FIG.3







SEEPAGE FORCE — BOUYANT FORCE — PORE PRESSURE FORCE

WITHIN A TRIANGULAR SECTION PERVIOUS BODY

FIG 4 — FIG.5 — FIG.6

triangular pervious body, the downward force $\sum_{i=1}^{n} h_i a_i$ should be added due to the small connected areas. This force is approximately equal to 15% of the bouyant force if $\sum_{i=1}^{n} a_i$ is considered as 15% of the gross area. As previously mentioned, concrete has about 12% voids and hence the bouyant force is only about 88% effective. The deduction of bouyant force due to voids and the increment due to a_i's cancel each other, thus it is concluded that for concrete the net bouyant effect is about 100% effective.

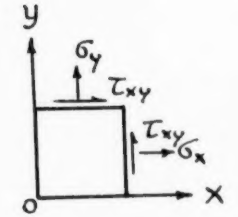
Combining the forces in fig. 4 and fig. 5, their resultants are shown in fig. 6, and they are called pore pressure forces.

In general, for any shape of pervious body, the equi-peizometric lines within the pervious body can be determined by electrical analogy of relaxation or graphical methods. After determining the equi-peizometric lines, the lines of equi-pressure follow readily.

Based upon this pore pressure conception, the loading condition of water acting on the concrete dam should be rearranged. For the water does not act entirely on the upstream face of the dam, it acts successively on the "space frame" structural body of concrete within the dam. The known pore pressure intensity distribution within the concrete dam gives the pore pressure magnitude and direction at any point within the concrete dam.

Stresses in Concrete Dams

By conceiving a concrete dam as a "space frame" which is under the actions of its own weight and the pore pressure forces resulting from the water dammed up behind it, the stresses in the concrete dam are analyzed. From the point of view of mechanics, the pore pressure force acting within a concrete dam constitutes a body force, which is measured as force per unit volume in the direction of pore pressure force gradient. The general equations of elastic equilibrium in two-dimensional problem have the form:(7)



STRESS ELEMENT
FIG. 7.

$$(\partial\sigma_x/\partial x) + (\partial\tau_{xy}/\partial y) = X \tag{10}$$

$$(\partial\tau_{xy}/\partial x) + (\partial\sigma_y/\partial y) = Y \tag{11}$$

where σ_x = normal stress in x-direction;

σ_y = normal stress in y-direction;

τ_{xy} = shearing stress

X, Y = body force in x and y-directions respectively.

The solution of equations (10) and (11) for a right triangular concrete dam subject to the action of pore pressure force within are:(8)

$$\left.\begin{aligned} \sigma_x &= r\,x/m \\ \sigma_y &= -\left(w' - \frac{2r}{m^2}\right) x/m + \left(w' - \frac{r}{m^2}\right) y \\ \tau_{xy} &= r\,x/m^2 \end{aligned}\right\} \tag{12}$$

where m = the ratio of width of the base to the height of the right triangular dam;

w' = unit weight of concrete in air minus unit weight of water.

And the corresponding solutions for the case that water does act entirely on the upstream face and the bouyancy is 100% effective are:(7)

$$\left.\begin{aligned} \sigma_x &= r\,y \\ \sigma_y &= -\left(w' - \frac{2r}{m^2}\right) x/m + \left(w' - \frac{r}{m^2}\right) y \\ \tau_{xy} &= r\,x/m^2 \end{aligned}\right\} \tag{13}$$

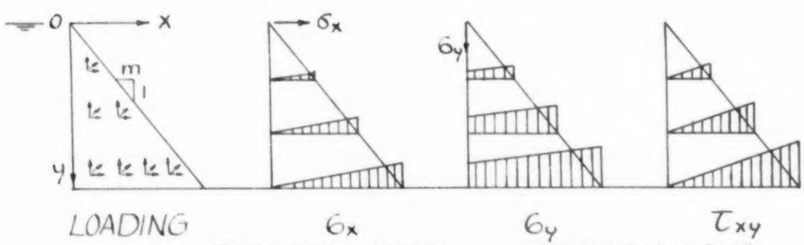
On comparing these two sets of solution, one finds that σ_y's and τ_{xy}'s are entirely identical and σ_x follows linear variation with x in one case, while

σ_x is constant in the other. However, at a certain level, the values of σ_x at downstream face for both cases are equal. Graphic representations of these two solutions are shown in figs. 8 and 9.

Effect of Drainage System on Pore Pressure Within Concrete Dams

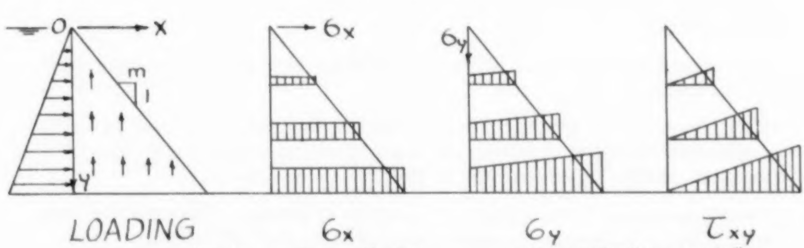
In order to reduce the pore pressure intensity within concrete dams, a drainage system consisting of a series of vertical drains is generally installed near the upstream face of the dams; thus the intensity of pore pressure at these vertical drains is usually assumed a certain percentage of the difference between maximum head water and maximum tail water.(5) The choice of the magnitude of the percentage is entirely based on experience, no definite reason has ever been given. However, as concrete being a pervious medium, those vertical drains form a series of wells. If proper boundary conditions are imposed on the fundamental triangular section of the dams, the resulting height of seepage face in hydraulics of well corresponds to the intensity of pore pressure at those vertical drains.

The writer used a combination of membrane and electrical analogies to investigate the height of seepage face of a single gravity well thoroughly in 1952.(9) It is suggested that the same technique could be adapted to find the seepage height at those vertical drains for dams with different height and also the most effective spacing of those vertical drains.



THE SOLUTION — EQUATION "12"

FIG 8



THE SOLUTION — EQUATION "13"

FIG. 9

CONCLUSIONS

1) The pore pressure force acting within concrete dams is composed of two components: one is the uplift force caused by bouyant effect; the other is seepage force resulted from the flowing water acting on the pervious medium —concrete.

2) By conceiving a "space frame" structural body, with 85% effective area according to Mr. Leliavsky's experimental result and 12% void as generally recognized, concrete is said to have 100% effective area because the downward water pressure on the solid portion of the area is approximately equal to the weight of water in the 12% void.

3) The horizontal water pressure resulting from the water dammed up behind concrete dams does not act entirely on the upstream face of the dams. In fact the resulting horizontal force acting on a dam is the sum of the horizontal components of the small seepage forces which act successively within the dam in the direction of seepage flow.

4) With fully bouyant effect, only the horizontal normal stress is different by assuming that the water acts entirely on the upstream face and that the water acts successively within the dam. The resulting horizontal normal stress from the former loading condition is uniform at a certain level and from the latter is a triangular variation. However, the horizontal normal stresses at downstream face for both cases are equal.

5) The pore pressure intensity at vertical drains near the upstream face and the most effective spacing of vertical drains can be well determined by electrical analogy in laboratory or other comparable analysis.

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7. "The Design of Dams," by A. Bourgin, Pitman Press, London, England, 1953, pp. 126-131. (Notations are properly changed to conform to general notations used in the theory of elasticity).

8. The solution is obtained by setting $X = \gamma/m$ (seepage force per unit volume) and $Y = w'$ and following the same procedures of solving equations (10) and (11) as treated in "The Design of Dams" by A. Bourgin, see Reference 7.

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Journal of the
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AMBUKLAO UNDERGROUND POWER STATION[a]

Andrew Eberhardt,[1] M. ASCE
(Proc. Paper 1598)

SYNOPSIS

This paper describes the design of the power features of the Ambuklao Project, a hydroelectric power development of the National Power Corporation of the Philippines. An underground power station was successfully constructed at a site which offered rather complex rock conditions. The outstanding feature of power station is the horizontal shaft setting of the generating units.

INTRODUCTION

The Ambuklao Project is located on the Agno River in Luzon in the Philippine Islands (Fig. 1). The nearest city is Baguio, the summer capital of the country, 28 miles away. The climate of the mountainous region is tropical with two rather sharply defined main seasons: wet and dry. Through the rainy season from June to November the irregular occurrence of typhoons affects greatly the intensity of rainfall. During a typhoon a rainfall was recorded at Baguio of 48 inches in 24 hours and of 105 inches in less than a week. Sudden rises and declines of the river level are characteristic of the rainy season flow with the maximum flow of record estimated at about 330,000 cfs. Minimum dry season flows of 350 cfs are common.

The project consists of: a central core, rockfill dam 430 ft high (one of the highest dams of this type), a gated overflow spillway with a steep chute and a flip bucket (Fig. 2), a low level outlet provided in one of the diversion tunnels, and the power features. The latter comprise: a tower intake, a pressure tunnel, penstocks, an underground valve chamber, and an underground power chamber with an access tunnel (Fig. 3 and 4). A long tailrace tunnel connects the draft tube outlets with the river. The transformers are placed above

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a. Presented at a meeting of the ASCE, New York, N. Y., October, 1957.

1. Chief Civ. Engr., Harza Eng. Co., Chicago, Ill.


PHILIPPINE ISLANDS
N
LUZON
BAGUIO
AMBUKLAO PROJECT
Philippine Sea
MANILA
China Sea
Sulu Sea
0 100 200 Miles

Fig. 1

DAM & SPILLWAY

Fig. 2

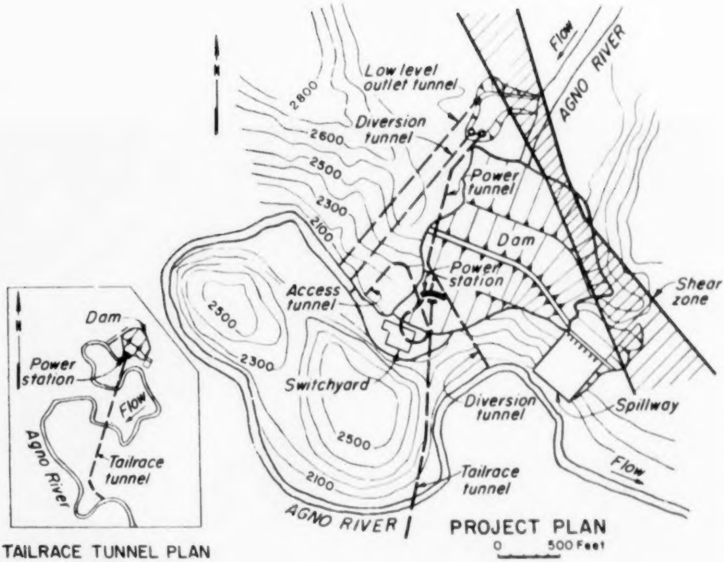
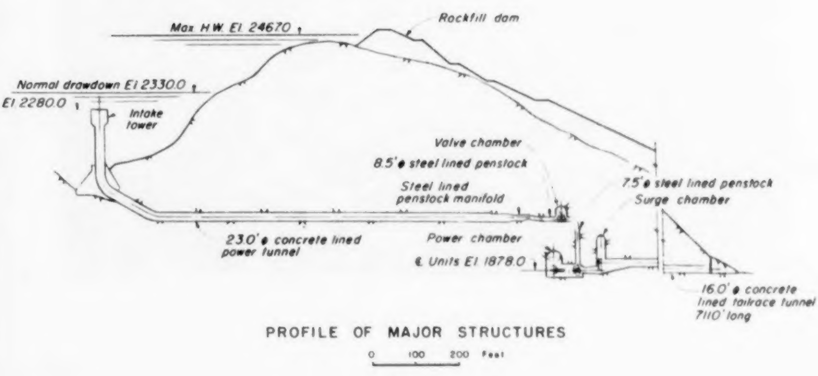


Fig.3



PROFILE OF MAJOR STRUCTURES

0 100 200 Feet

Fig. 4

ground in the switchyard which adjoins an administration building.

The rated output of the power station is 75,000 kw under a net head of 505 ft. The transmission system consists of a 220 kv line to Manila 150 miles long and a 69 kv line to San Fernando, La Union, 40 miles long.

Geology

Because of the underground location of the power station the geology of the site was of primary importance and strongly influenced the planning of the power facilities.

The principal rocks in the vicinity of Ambuklao are diorite and metamorphosed rocks. The first is a medium grained rock similar to granite; the latter consist of metamorphosed andesites and interbedded sedimentary rocks. The andesites and the sedimentary rocks were laid down respectively as lava flows and water deposited sediments in Tertiary time. After deposition and consolidation into rock they were subjected to mountain building forces and were injected by molton magma which solidified to form irregular tongues of diorite and other igneous rocks. In these processes the lava (andesites) and sedimentary rocks were metamorphosed and fractured.

Due to the intense fracturing the metamorphic rocks are weathered to a great depth, a condition often encountered in the humid tropics where rainfall is heavy. The surface appearance of these rocks at Ambuklao gives the impression of weak and unstable material and is so unfavorable that, at one stage of planning, it caused temporary abandoning of the right abutment as the location of the power facilities.

The same rocks, however, when seen inside a deep tunnel are moderately strong, hard though brittle, and their numerous fractures are tight.

As a result of the intrusions of the diorite, the contact between the two principal types of rock is extremely irregular, usually tight but with a tendency for the rocks to be more fractured near the contact lines. The diorite, where it is unweathered and unfractured, is a strong, hard rock superior to metamorphics. A number of faults intersects the rocks in the vicinity of the project.

Underground Location of Power Station

Initial studies indicated that an underground type power station was much more advantageous at Ambuklao than a conventional power station located above ground. The winding river provides an opportunity for gaining considerable additional head by constructing a long free flowing tailrace tunnel. The tunnel bypasses the obstacles offered by the steep mountain ridges and deep valleys and emerges 5 river miles downstream of the plant to return the water to the river.

A conventional power plant located as to develop the same total head would require a long pressure tunnel, exposed penstocks, and a surge tank. If the conventional power station were located at the dam site the available head would be reduced by about 180 feet with a corresponding increase in the cost per kilowatt of installed capacity. The underground location also offered safety from landslides and rockfalls, a greater war time security, and a greater freedom from tropical rains and floods during the construction period.

The major problem was one of locating rock of quality suitable for an underground installation and particularly for the power chamber which, due to its large span and vertical sides, required better rock than the circular or

horseshoe shaped tunnels. This task was complicated by the deep weathering and the irregular character of rocks. There was also some concern about creep in metamorphic rocks which is prevalent to unusual depths in this region of very rugged mountains, with steep slopes, deep weathering, and extremely heavy rainfalls. Creeping of rock has been observed in the local mines to depths of 200 feet.

Beside core drilling, needed information regarding rock was provided by an old prospecting tunnel downstream of the site and by an exploratory tunnel in the west abutment. In addition, the first of three diversion tunnels required for the construction of the dam, was driven quite early in the construction schedule providing valuable data on the behaviour of the metamorphosed rocks in the west abutment.

A large factor in the selection of the final location for the underground power station was the presence of a major fault upstream which runs diagonally across the river and cuts into the left or east abutment of the dam (Fig. 3). The rock in the fault zone from 200 to 300 ft wide is sheared and disintegrated with much of it decomposed to clay. It was feared that an earthquake shock originating within the fault could rupture tunnels built across the fault and severely damage any underground installation built close to it, even if placed in good rock. Difficulty of driving tunnels across the shear zone was also considered. As a result the power station was placed in sound diorite disclosed by drilling, on the right bank, approximately 200 ft. underneath the downstream toe of the dam and away from the fault, with none of the underground water conductors intersecting the latter. The downstream location of the power station also had the advantage of utilizing one of the diversion tunnels as a power conduit and thus shortening the length of the tailrace tunnel.

As it developed, no unusual difficulties were encountered during the extensive tunneling work required at Ambuklao, with the exception of a cave-in in the power chamber as described further on.

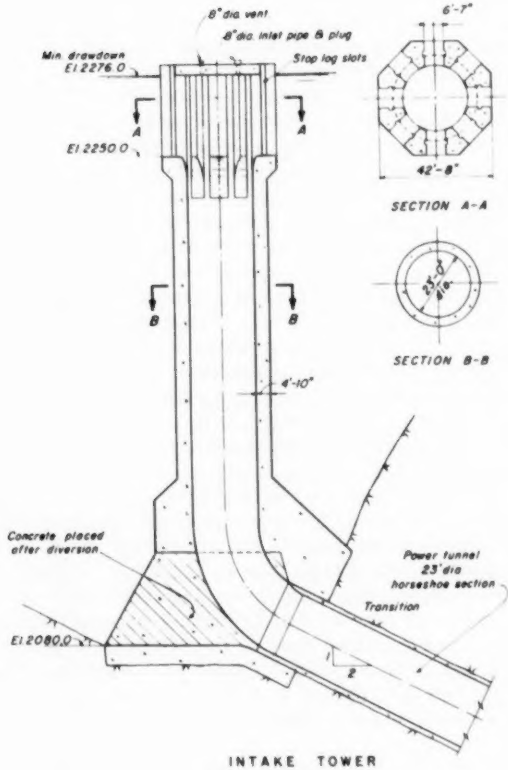
Power Intake

The initial studies included an intake built into the steep hillside. The idea, however, had to be abandoned because of large open cut excavation required in overburden, deeply weathered rock and the possibility of slides.

The intake was given the form of a tower with inlets at the top, founded on rock at the toe of the hillside and built sturdily enough to defy earthquakes common to the area. The depth of the reservoir at the intake location (380 ft) presented a problem. The cost of a tall structure reaching above the water level and capable of resisting earthquake shocks, was considered excessive. The decision, therefore, was made to shorten and submerge the tower, thereby sacrificing intake accessibility and eliminating intake closure gates, but achieving a considerable saving in the project cost.

As built, (Fig. 5), the top of the intake tower is 184 ft below the full reservoir level and 54 ft below the normal minimum drawdown level. Only in an extremely dry year the top of the intake may emerge from the reservoir depths. Normally, divers will be required to place stop logs and an air vent extension when unwatering of the tunnel is necessary.

It is felt that with a properly built tunnel, chances for its sudden collapse or blockage are remote. The main disadvantage of the arrangement adopted at Ambuklao is a probable longer outage due to the time required to unwater the tunnel for inspection or maintenance.



INTAKE TOWER

0 10 20 30 Feet

Fig. 5

Power Conduits

The river was diverted during the construction period through three tunnels. Two of the three tunnels were designed to serve as permanent water conductors: one provides a low level outlet for drawing down the pool for maintenance and inspection of the power intake system and for supplying water to downstream installations in case of a shut down of the Ambuklao station; the other functions as the power tunnel.

The power tunnel is a 23 ft diameter horseshoe tunnel approximately 1200 ft long. The whole length of the tunnel is lined with concrete which is reinforced in the areas of weak rock. During the diversion a temporary inlet was provided in the intake tower base. After the diversion the tunnel was converted into a pressure tunnel by filling the opening in the intake base with concrete, plugging the tunnel near its downstream end with a concrete bulkhead and constructing a penstock manifold, ahead of the latter.

The manifold (Fig. 6), consisting of successive cylindrical and conical reducing sections and branch connections, distributes the flow between three penstocks. The penstocks, 8.5 ft in diameter, run horizontally to the valve chamber, then make a 90° vertical turn and continue straight down to the spiral case inlets. The vertical penstocks are 7.5 feet in diameter.

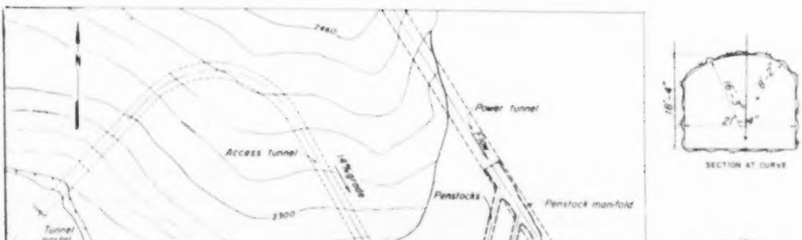
The manifold and the penstocks are lined with steel plate embedded in concrete. The following factors were considered in determining the plate thickness: internal pressure (static plus waterhammer), external pressure with penstock empty (ground water and grouting pressures) and minimum handling thickness. The steel liner was assumed to take the full internal pressure without any support from the surrounding concrete and rock. The allowable working stresses, however, have been raised above those permissible for exposed penstocks. At the upstream end, the manifold liner has been designated for a working stress equal to the yield point (27,000 psi) while the working stress in the penstock was held down to 2/3 of the yield point or 18,000 psi. The more conservative approach, assumed in the latter case, was dictated by the proximity of the valve and power chambers. The forces caused by the discontinuity of the manifold liner at the penstock branches were assumed to be resisted by rock with only nominal stiffeners provided at the plate intersections. ASTM 285-50 T, grade B firebox quality steel was specified. Grade B, being more ductile than Grade C, was considered more desirable for this particular application.

The horizontal sections of the penstocks were provided with grout and vent holes for grouting the invert of the liner and for grouting the concrete-to-rock contact along the crown. The vertical sections of the penstocks were grouted only within 50 feet of the power chamber.

The velocity in the penstocks is 15.1 fps at rated discharge (667 cfs per unit) and 17.3 fps at the maximum discharge. The maximum pressure rise due to waterhammer is 23% of the static head.

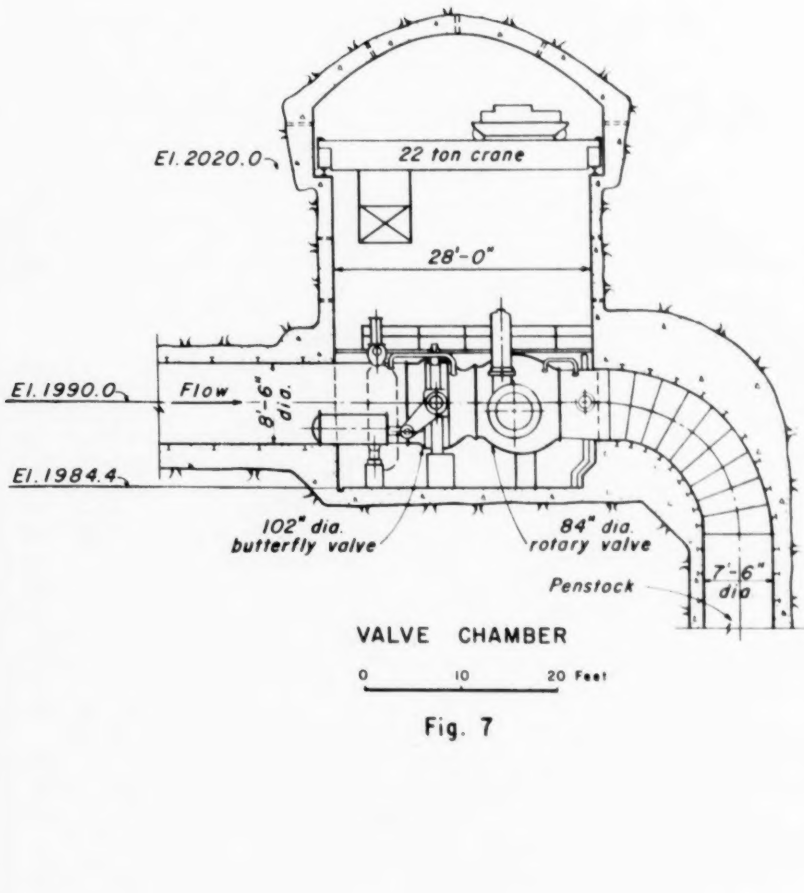
Valve Chamber

The penstock valves were placed in a separate underground cavern located approximately 60 ft above the roof of the power chamber. The valve chamber houses six high head valves and a bridge crane of 22 tons' capacity for erection and maintenance of the valves (Fig. 7). Access is provided by a short tunnel branching from the power station access tunnel. The roof of the cavern is supported by a concrete arch poured directly against the rock surface. The



POWER STATION ACCESS TUNNEL

Fig. 6



El. 2020.0
22 ton crane
28'-0"
El. 1990.0
Flow
8'-6" dia.
El. 1984.4
102" dia. butterfly valve
84" dia. rotary valve
7'-6" dia
Penstock
VALVE CHAMBER
0
10
20 Feet

Fig. 7

walls are lined with concrete anchored in the rock. Weep holes in the roof arch and in the wall lining provide relief from hydrostatic pressure that could develop behind the lining due to the proximity of the power tunnel (the steel liner ends about 130 feet upstream of the valve chamber).

Each penstock is equipped with two valves in tandem: a 102-inch diameter butterfly valve and a 84-inch diameter rotary valve (Fig. 8). The butterfly valve serves as a guard valve for the rotary valve which is used in normal service, i.e. when it is necessary to shut down the unit or unwater it for maintenance. In view of the high head it was not desirable to rely on wicket gates alone whenever the unit is off the line.

The double valve arrangement was selected for two reasons: to avoid an outage of the whole plant in case of repairs to a single service valve and to assure extra means of closing the flow of water in case of an emergency, such means not being available in the form of a gate in the submerged intake tower.

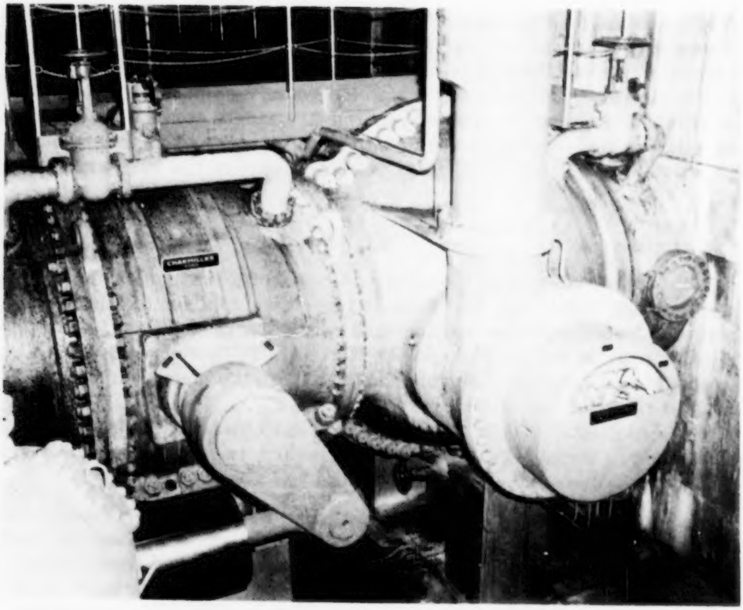
Power Chamber

The outstanding feature of the power station is its horizontal shaft setting of the generating units (Fig. 9). This setting which appears to be particularly advantageous in underground applications, was first used by the Harza Engineering Company at the Guayabo power station in El Salvador, C. A. Its merits and disadvantages have been discussed in detail in the description of this project in a recent issue of Water Power.*

Since the Ambuklao design has been patterned closely after its prototype, Guayabo, most of the observations and conclusions regarding the latter are also valid in relation to the Ambuklao design and are quoted below:

> "The horizontal shaft setting was intended first to reduce construction cost in terms of power chamber excavation, substructure concrete, and assembly time. Since the width and height of the power chamber can be considerably reduced with the horizontal settings and the spacing of units is no greater than with the vertical shaft setting, very considerable savings in excavation were realized. A recent comparison of this Guayabo design with those of nine other modern vertical shaft type underground power developments, using the generator diameter as the critical parameter, indicates that the Guayabo station requires only 60% as much excavation as the average of the others. The savings in substructure concrete appear to be at least as great. Reduction in construction time resulted from the reduced work quantities and also from the fact that, with the horizontal setting, both the generator and the turbine can be assembled at the same time."
>
> "A final saving in construction cost comes from substituting the horizontal straight axis conical type draft tube for the more complicated elbow type draft tube used with a vertical shaft setting."
>
> "The second objective of the horizontal shaft arrangement was to obtain greater hydraulic efficiency by eliminating the bend which usually occurs ahead of the spiral case in a vertical shaft setting and eliminating the elbow bend in the draft tube."
>
> "The horizontal setting has the following disadvantages which must be taken into account:

*"Guayabo Hydroelectric Power Project," by R. D. Harza and T. Zowski, Water Power, May 1957.



PENSTOCK VALVES
Fig. 8

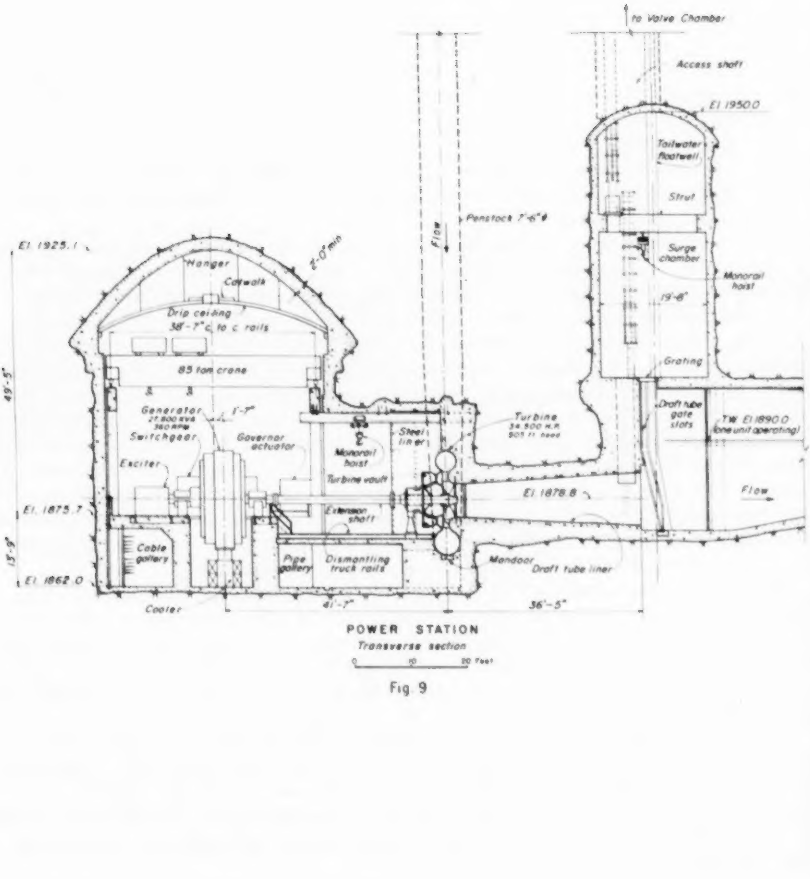

to Valve Chamber
Access shaft
El 1950.0
Tailwater floatwell
Strut
Penstock 7'-6"φ
Flow
El. 1925.1
2'-0" min
Hanger
Catwalk
Surge chamber
Monorail hoist
Drip ceiling
38'-7" c. to c. rails
19'-8"
85 ton crane
Grating
49'-5"
Generator
27,800 KVA
360 RPM
Switchgear
Governor actuator
Monorail hoist
Steel liner
Turbine
34,500 H.P.
905 ft. head
Draft tube gate slots
TW El 1890.0 (one unit operating)
Exciter
Turbine vault
El 1878.8
Flow
El. 1875.7
Extension shaft
13'-9"
Cable gallery
Pipe gallery
Dismantling truck rails
Mandoor
Draft tube liner
El 1862.0
Cooler
41'-7"
36'-5"
POWER STATION
Transverse section
0 10 20 Feet

Fig. 9

(a) The turbines are not within reach of the main power station crane, therefore, special handling devices must be provided in the turbine vaults. In the Guayabo station two hand operated monorail hoists are provided for this purpose.

(b) The horizontal setting is not favorable to frequent use of the generating units as synchronous condensers for improving power factor and voltage regulation. The horizontal draft tube, with runner below the tailwater level to assure adequate submergence of the draft tube outlet, does not permit ready evacuation of the water around the runner by admission of compressed air as is done in vertical turbines. To permit the runner to rotate in air, the draft tube gate would have to be closed and the water pumped out."

The conclusion was reached, similarly as at Guayabo, that the disadvantages are greatly outweighed by the advantages of the horizontal shaft setting (synchronous condenser operation was not required at Ambuklao). Since then the experience acquired during the constructions of both power stations and the results of the turbine and generator acceptance tests at Guayabo confirmed the validity of this conclusion.

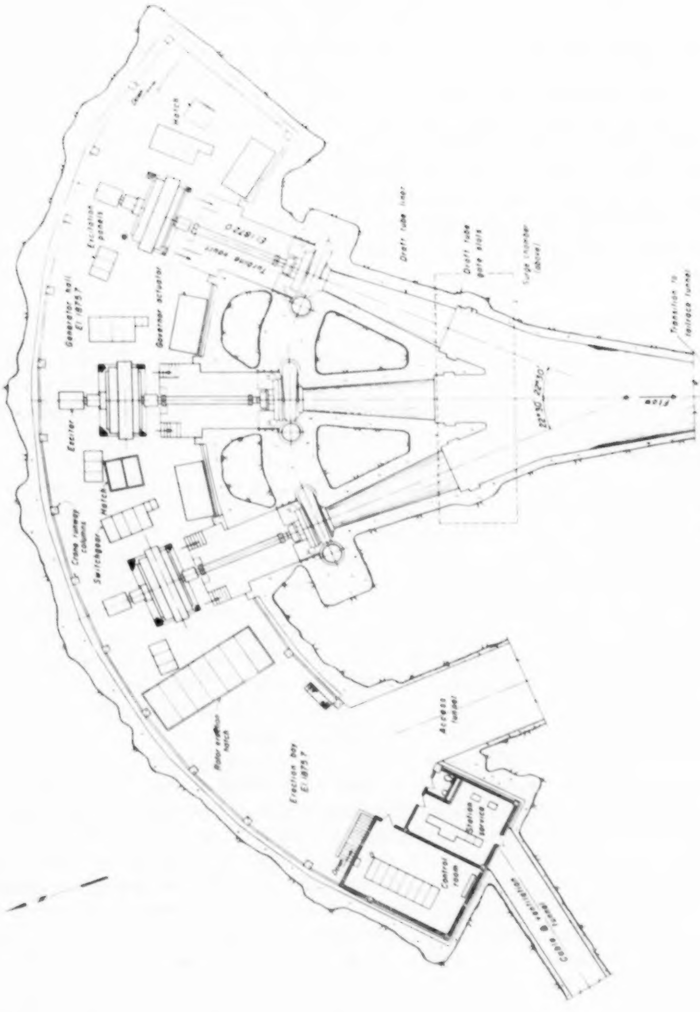
The Ambuklao power chamber curves in plan with the turbine-generator units arranged radially and with the draft tubes converging gradually toward the tailrace tunnel (Fig. 10).

The turbines are housed in individual vaults carved out in rock in the downstream wall of the generator hall. Switchgear and governor actuators are placed on the generator hall floor, leaving little unused space between the individual pieces of equipment. The erection bay and the control room are located at the same level as the main equipment, at the access end of the generator hall. The compact design of the power chamber stands out particularly in its transverse section: the low cavern contains only a shallow basement below the generator floor, built around the generator foundations, with a cable gallery on one side and a pipe gallery on the other. The basement extends the full length of the generator hall, the erection bay and the control room. The sump pumps, air compressors, and CO_2 equipment are located in the basement using up the remaining space between the generator foundations. The basement area below the erection bay and the control room is occupied by the machine shop and the spreading room, respectively.

The horizontal shaft setting requires different handling of generator and turbine parts than its more conventional vertical unit counterpart. The required clearances were carefully studied in order to achieve the primary goal of keeping the volume of underground excavation to a minimum. The height of the crane rail was determined by the requirements of the rotor assembling operation (Fig. 16) and by the height of the turbine vaults.

A parabolic concrete arch poured directly against the rock supports the roof of the cavern. A drip roof of corrugated transite sheets suspended from the arch on galvanized steel hangers keeps the interior of the chamber dry (the concrete arch has drain holes drilled in the rock above to prevent a build-up of seepage pressure). The crane runway beams and columns are of reinforced concrete.

As the excavation progressed, areas of poor rock appeared in a few locations. A major cave-in involving approximately 400 cy of rock (about 250 of which came from outside the excavation line) occurred in the roof where the rock was the weakest, fortunately without causing any casualties. The



POWER STATION
PLAN

Fig 10

warning came in the form of a sudden increase in the inflow of water which caused the tunneling crew to move away in time. The flow of water stopped very soon after the cave-in, apparently indicating a temporary build-up of hydrostatic pressure inside the rock fissures relieved by the cave-in.

The reinforced concrete arch supporting the cavern roof is of 2 ft minimum thickness except for the cave-in area where the arch thickness was increased to 4 feet. The space above the concrete arch at the cave-in was filled with concrete and grouted. The walls of the cavern are strengthened with heavy concrete lining anchored deep into the rock and punctured with weep holes. Due to the irregular shape of the overbreak the thickness of lining varies considerably. The turbine vaults are also lined with concrete.

A grouting program was planned in advance and carried out in the power chamber and in the valve chamber to seal off and consolidate the rock. The concrete-rock contact in the roof arches of all underground chambers and in the tunnels was also grouted.

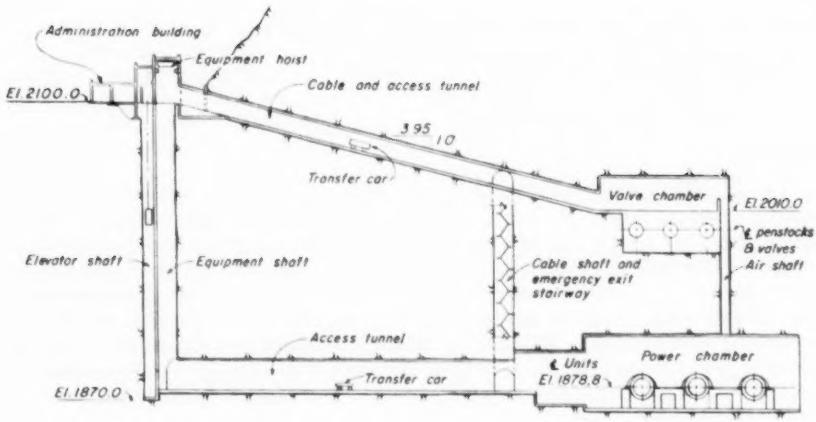
Power Station Access

Preliminary studies included a comparison between an inclined access tunnel and a vertical access shaft. Due to the considerable depth of the power station level below the switchyard and the access area (over 200 ft) the vertical shaft scheme appeared substantially less expensive and for this reason it was incorporated in the design drawings (Fig. 11).

The shaft was made large enough to admit turbine and generator parts and also to house a passenger and freight elevator. Hoists for handling equipment and for the elevator were placed inside the administration building which straddled the shaft opening. Since the topographic conditions prevented locating the building and the shaft adit directly above the power station, a short horizontal tunnel was required to connect the shaft bottom with the underground power chamber. Another tunnel inclined at 25% provided access to the valve chamber. The latter was connected with the power chamber below by a vertical shaft equipped with a stairway. Both the inclined tunnel and the shaft also housed power and control cables and provided means of an emergency exit from the power chamber in case of an elevator failure.

Before the start of construction, however, the contractor whose staff included Swedish tunneling engineers, proposed to build an inclined access tunnel in lieu of the vertical shaft. He reasoned that the cost of additional excavation involved in building a long tunnel would be more than offset by the convenience of access provided by a vehicular tunnel during construction and by expediting of all underground work in connection with the power chamber and the valve chamber. Since the contract was of the "target estimate" type, the owners were to participate in any overall saving at the completion of the job. This feature made the contractor's proposal more attractive and his idea was accepted, after some studies and modifications.

The tunnel as built is nearly 1/3 mile long. It winds and spirals downward toward the power station at a steep 14% grade providing only a single lane on straight ways and two lanes on curves (Fig. 6). The double width was made sufficient to accommodate two Koehring dumptors. The lower portion of the access tunnel was placed away from the pressure power tunnel in order to reduce seepage inflow and also to take advantage of better rock in this location. For reasons of economy the access tunnel was left initially unlined and unpaved. Only about one-third of the tunnel length required timber supports and



VERTICAL ACCESS SHAFT SCHEME

Fig. 11

lagging in the areas of poor rock. The inflow of seepage water, however, was considerable after the reservoir filling. The seepage was attributed to the proximity of the low level outlet tunnel whose almost whole length is subjected to the full headwater pressure. A grouting program was planned accordingly and soon after the start of the grouting operations the inflow was reduced by one-half.

No figures are available at present to determine the actual cost of construction of the access tunnel. Furthermore additional work of grouting, lining, and paving is still to be carried out. It is doubtful, however, whether it will ever be possible to make a reliable cost comparison between the original scheme of a vertical shaft and the tunnel since there is no way of determining the effect of the latter on the overall cost of work carried out inside the underground chambers. It can only be stated that the advantages of a vehicular tunnel permitting direct truck access should always be given sufficient weight in any comparative studies. Two other underground power stations designed by Harza Engineering Company: Guayabo in El Salvador and Maithon in India, both have such tunnels. In each case, however, the tunnel length is considerably shorter than at Ambuklao and the grade is somewhat less steep.

An emergency exit from the power chamber is provided by the cable and ventilation tunnel (Fig. 6) sloping up at a 76% grade and equipped with a stairway.

Surge Chamber

The presence of a long tailrace tunnel, which flows full during the high tailwater periods, required the construction of a surge chamber at the draft tube outlets (Fig. 9). The surge chamber improves stability of operation of the units and reduces the pressure rise in the tailrace tunnel and draft tubes caused by a sudden load demand. The surge chamber also serves as an operating chamber for the draft tube gates, the draft tube deck constituting the bottom of the surge tank. The surge chamber is of the restricted orifice type with the draft tube gate slots providing the orifice openings. The openings are covered with steel gratings and their area can be adjusted by means of cover plates bolted to the gratings.

The height of the surge chamber was determined by the upsurge following the initial drop in water surface caused by full load rejection on three units, with the wicket gates closing from over-gate (110%) to speed-no-load position in the minimum governor closing time, and with the tunnel flowing full. The negative wave formed by the load rejection travels for some distance downstream, diminishing until it disappears and the empty space in the tunnel starts filling again due to tailwater back-pressure. The flow in the tail tunnel is then reversed and water rushes back into the surge chamber, together with the speed-no-load flow from the turbines. The surge level in the chamber rises until it reaches its maximum, El. 1950.0 or 5 ft below the crown of the chamber.

Because of the interconnection existing between the surge chamber and the generator hall (through the valve chamber and the access tunnel), a study also was made of the highest probable surge elevation caused by load demand occurring after load rejection at the right moment for the peaks of the rejection and demand surge curves to coincide and to become additive.

It was found that combined upsurges, if such a rather improbable case did occur, would rise quickly into the surge chamber access shaft and then

disperse rapidly as they spread into the valve chamber through the interconnecting tunnel, but they would not flood the generator hall.

The access to the surge chamber is from the valve chamber: first along a short inclined underground gallery, then down a vertical shaft which pierces the surge chamber roof. The shaft contains an access ladder and the tailwater float well.

The monorail provided for handling the draft tube gates is hand operated because the surges will submerge it from time to time. The presence of surges also imposes a restriction on the accessibility of the draft tube deck and the use of the gates. The unit inspection and maintenance work has to be scheduled during the low water season (from December to June) when the tunnel flows free, unless the reservoir is drawn down and no spill occurs. This restriction paid off in a considerable saving due to the lower height of the surge chamber and due to the elimination of a high level deck or platform for operation of the draft tube gates at all times. A high level deck would have also required a special mechanism for removal of heavy gratings covering gate slots.

The access gallery also serves as an oversize drain or overflow in case of a penstock rupture near or inside the valve chamber or a penstock valve failure. Such an overflow was considered necessary because of the interconnection existing between the valve chamber and the power chamber through the main access tunnel. Another possibility of a structural failure taken under consideration was sudden blocking of the tailrace tunnel by a cave-in caused by tectonic movements. Such a possibility, even if remote, could not be entirely disregarded in a region of earthquake. Blocking of the tailrace tunnel, even if only partial could result in backing up of water from the surge chamber into the valve chamber and from there into the power chamber until all the units were shut down. It was preferred not to rely during such an unusual emergency on the operators alone, therefore, an automatic float alarm was provided near the valve chamber floor level. The valve chamber floor is sunk 17 ft below the level of its access tunnel in order to minimize the possibility of water overflowing into the tunnel and continuing down to the generator hall during any of the two emergency situations considered above.

Tailrace Tunnel

The tailrace tunnel is of circular cross-section 16.4 feet in diameter. It is 7110 feet long and discharges into the Agno River about 5 river miles downstream of the damsite. A converging tailrace transition provides a smooth water passage from the draft tube outlets to the circular tunnel. The invert of the transition slopes upward from the draft tubes to assure submergence of the turbines at low tailwater.

With the tunnel flowing 0.9 full, the velocity under full plant discharge of 2290 cfs is approximately 11.5 fps. The tunnel is lined throughout with concrete. The type of the tunnel, its size and the use of lining were determined on the basis of an economic study. The tunnel runs through comparatively good rock and its construction was performed without difficulties.

Turbines

The turbines are of the horizontal shaft overhung runner Francis type, housed in cast steel spiral cases. The spiral cases are embedded in concrete in individual vaults excavated in rock downstream of the generator hall

(Fig. 12). The turbines discharge into horizontal conical draft tubes which are steel lined and embedded in concrete. Each turbine has a combined radial and thrust bearing mounted on the head cover and supported vertically on a short pedestal. The turbine shaft is connected through an intermediate shaft section to the generator shaft.

The turbines have a rated capacity of 34,500 horsepower each at full gate opening when operating at 360 rpm under a net head of 505 feet to match the generator rated capacity of 25,000 kw. Each turbine also has a guaranteed capacity of 21,000 horsepower at full gate under the minimum operating net head of 380 feet. The turbines are designed to operate with good efficiency at net heads between 380 and 572 feet.

The runner is a single piece steel casting and has a discharge diameter of 66.9 inches. Replaceable wearing rings of carbon steel are provided on the runner periphery and adjacent stationary parts, where small running clearances have to be maintained. The distributor has replaceable wearing plates adjacent to the wicket gates, and the discharge ring has a removable liner.

The turbine shaft of forged steel revolves in a combined guide and thrust bearing of the self-lubricating type. Both the horizontal journal bearing and the flexible shoe thrust bearing are mounted in a common bearing shell which is spherically mounted in the bearing housing. This feature permits self-alignment of the bearing following shaft deflection. Both bearings are lubricated with oil collected and cooled in a reservoir located underneath the bearing housing. A starting oil pump is also provided.

The spiral case with integrally cast stay ring has an inlet diameter of 63 inches and is made in two sections with bolted radial flanges. It is designed for a hydrostatic pressure of 300 psi.

The head cover is a single piece steel casting. The main turbine shaft seal mounted on the head cover consists of two sets of spring loaded carbon rings which seal radially against a sleeve on the shaft. Drained labyrinth seals are also provided between the runner crown and the head cover. A head cover drain connected with the draft tube by a 150 mm pipe around the spiral case relieves water pressure on the head cover and reduces hydraulic thrust on the runner. The head cover is designed to withstand a maximum pressure head of 74 feet which may occur in the draft tube due to a surge in the surge chamber under flood conditions.

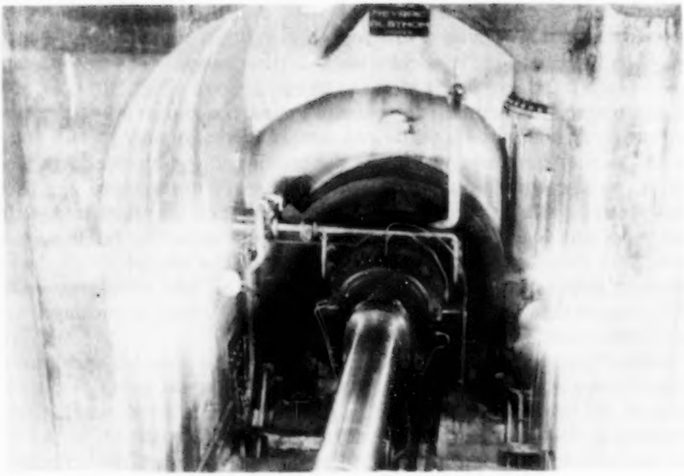
The gate operating ring is connected to two servomotors mounted vertically on base plates embedded in the concrete floor of the turbine vault. Access to the spiral case is provided through a manhole in its bottom part exposed in the turbine vault basement.

Each turbine is equipped with an oil pressure operated speed governor of 87,000 ft-lb capacity, sufficient to open or close the turbine wicket gates in four seconds. The governors are of the actuator type with a mechanical speed responsive element consisting of flyballs driven by an electric motor which receives its supply from a permanent magnet generator connected to the generator shaft.

The components of the governor which are located on the generator hall floor are enclosed by metal panels, with indicating instruments and controls mounted thereon.

Turbine Erection and Dismantling

Due to the horizontal shaft setting, handling of the turbine parts is quite



TURBINE VAULT
Fig. 12

different from that used in a conventional vertical setting (Fig. 13). As the turbine vaults are beyond the reach of the bridge crane a monorail track beam has been suspended from the roof arch of each turbine vault and a hand operated monorail hoist of 15 ton capacity has been provided. The hoist can be shifted from one vault to another with the help of the bridge crane.

The runner and shaft assembly is first completed in the erection bay, then deposited by the bridge crane at the mouth of the turbine vault at an angle (in plan) in order to clear the generator downstream bearing (Fig. 13a & b). The monorail hoist is then used together with the crane to lift the runner and shaft assembly at each end. The assembly is raised sufficiently to clear the top of the generator bearing, moved a few feet downstream and rotated to an axial position, then lowered onto a special dismantling truck. The truck travels on two steel beams partially embedded in the vault floor (Fig. 13c & d). Rollers bearing against the underside of the beam upper flanges prevent the truck from tipping over under the unbalanced weight of the runner at one end.

Once the runner assembly rests on the truck and the shaft end is tied down temporarily to prevent tipping, both hoist hooks are disengaged and the truck is pushed toward the spiral case. The runner is then inserted inside the stay ring (Fig. 13e) and the truck is removed while the monorail hoist supports the shaft end. The remaining turbine parts such as: the wicket gates, head cover, main bearing, gate operating ring, bearing pedestal, and servomotors are erected in succession with the help of the monorail crane and temporary blocking of the shaft end when necessary (Fig. 13f).

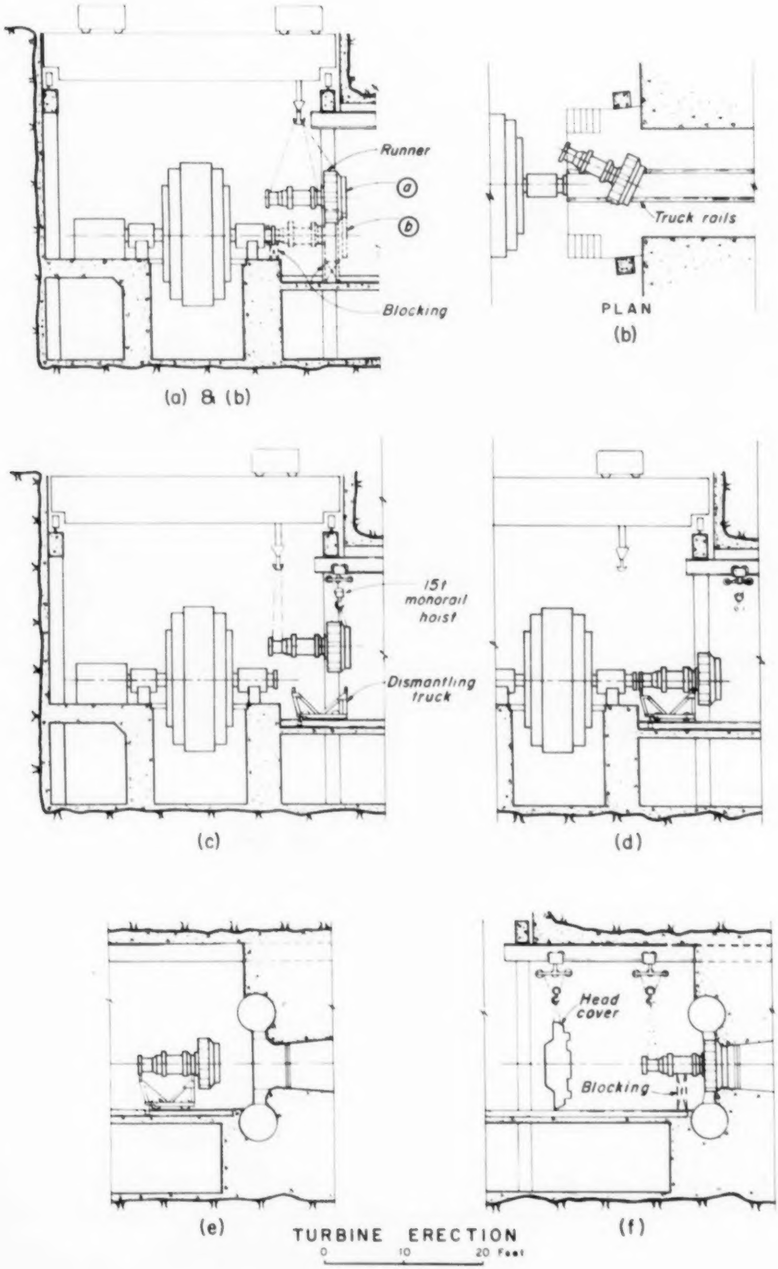
The dismantling operation can be performed quite simply, with the help of the dismantling truck, by removing as a unit the internal parts of the turbine consisting of the head cover, the wicket gates, the combined bearing, the shaft, and the runner. This particular operation begins with removal of the bearing pedestal, disconnecting the gate mechanism and removal of the operating ring, then the dismantling truck is brought underneath the turbine bearing and the turbine shaft end is fastened to the truck. With the truck supporting the shaft and the runner, the extension shaft can be removed without the runner coming to rest on the labyrinth seals. The next step consists of removing the bolts holding the head cover to the spiral case. Once this is done the whole assembly can be withdrawn from inside the spiral case into the turbine vault where further dismantling operations or inspection can take place.

The erection of the embedded turbine parts required two monorail hand operated hoists with the second hoist traveling either on the permanent track under the vault roof or on a beam mounted inside the spiral case excavation. Fig. 14 illustrates this method in application to the upper half of the spiral case.

Generators

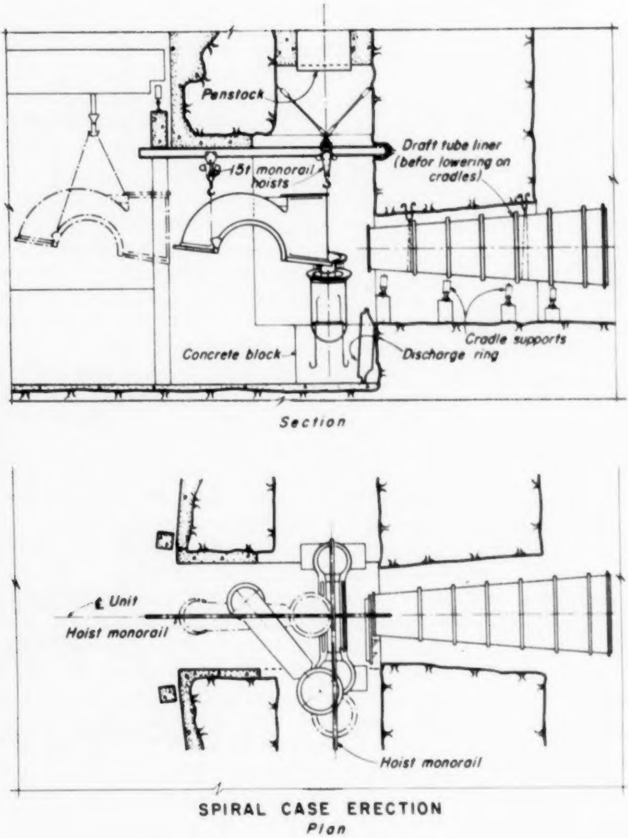
The three generators are each rated 27,800 kva, 0.9 power factor, 13,800 volts, 3-phase, and 60 cycle at 360 rpm. The generators are totally enclosed, horizontal shaft, synchronous machines, each with two radial type bearings supported on short pedestals (Fig. 15). The generator foundations are in the shape of rectangular boxes or pits built of reinforced concrete, and resting directly on rock. In addition to the lower portion of the generator, they house the water-cooled heat exchangers.

Each generator is excited by a direct-connected, horizontal shaft d-c generator mounted on the floor upstream of the generator. The permanent



TURBINE ERECTION

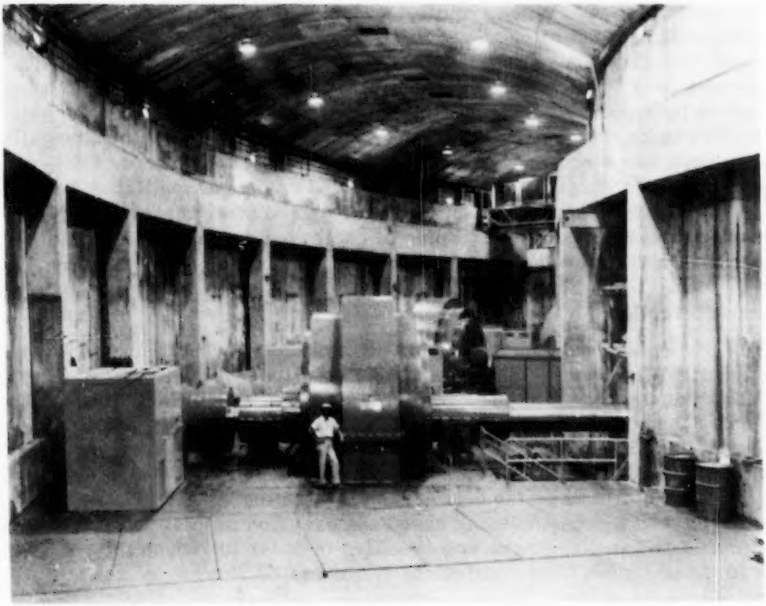
Fig. 13



SPIRAL CASE ERECTION

Plan

Fig. 14



GENERATOR HALL
Fig. 15

magnet generator, which provides current for driving the governor flyballs, and the overspeed switch are mounted on the exciter shaft extension.

The rotor, 141 inches in diameter, is of the "removable pole piece" design which permits removal or assembly of pole pieces with the shaft and the rotor resting in the bearings in the normal position and with the stator completely assembled. An alternative solution, consisting of shifting the stator horizontally in order to permit access to the rotor, was also considered. The "stator shift" however, which required an increase of over 3 feet in the power chamber width, did not offer any additional advantages to offset the increased expense. The stator is split in two halves along its horizontal centerline.

The generator terminals are connected to the metal clad switchgear located adjacent to the generator with short cables extending through generator pit walls. The 15 kv rubber insulated copper power cables enter the cable gallery opposite each unit switchgear, continue down the gallery and across the spreading room, then climb the steeply inclined cable and ventilation tunnel. Flexible band cable supports mounted on galvanized steel brackets grip the cable and prevent slippage or creeping of cables which could be caused by expansion and contraction due to load cycles.

Generator Erection

The rotor spider and shaft assembly brought to the erection bay in a horizontal position, is lowered into a special hatch (Fig. 16), then rotated around one end of the shaft to bring the assembly to a vertical position, required for assembling the rotor rim and the pole pieces. After the rotor assembly is completed, the rotating maneuver is repeated in reverse and the rotor with the shaft are brought into a horizontal position in which they are transported along the generator hall by the bridge crane. A lifting beam with a special lifting device and a cradle bearing have been provided to facilitate the rotating operation. A two trolley crane was specified in order to reduce the required height of the chamber. Each half of the stator can be handled by a single trolley.

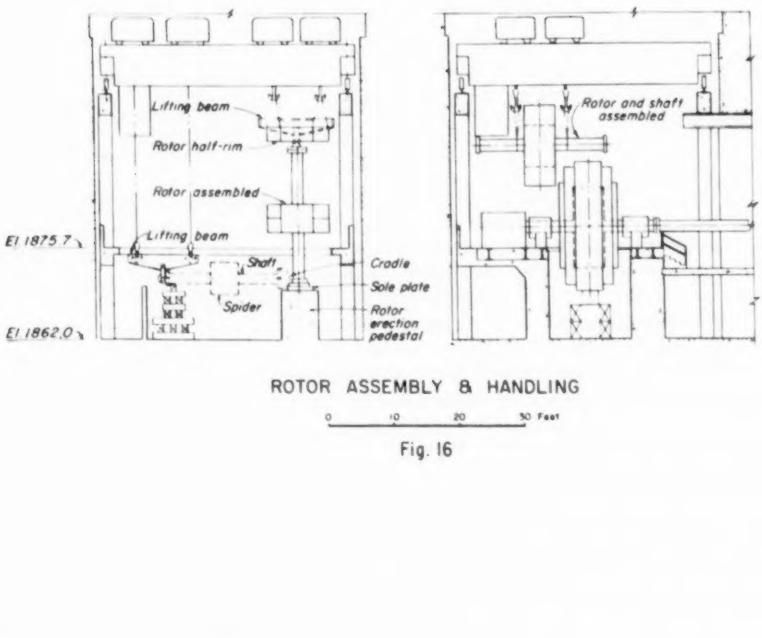
Transformers

The transformers and the switching equipment are located above ground, in the switchyard, on the right bank near the toe of the dam. The main power cables connect to the 13.8 kv bus structure erected next to the entrance to the cable and ventilation tunnel. There are two 32,000 kvs 13.8/220 kv three-phase transformers and two 20,000 kva 13.8/69 kv three-phase transformers whose high voltage sides are connected respectively to two transmission lines; the 220 kv line to Manila and the 69 kv line to San Fernando. The station service power is provided by two transformers each rated 750 kva and 480 volts.

Station Auxiliaries

The auxiliary mechanical and electrical systems, which provide the necessary station service, do not represent any particular departure from those used in more conventional above-ground power plants, with a possible exception of the ventilating and drainage systems which are of special importance in an underground installation.

In view of the tropical climate at Ambuklao, large quantities of air were considered necessary for cooling purposes and also for evaporation of a



ROTOR ASSEMBLY & HANDLING

Fig. 16

portion of the possible seepage through the rock walls. In most cases, the assumed air change values are at least twice the values accepted for above-ground powerhouses.

The station is ventilated by induced-draft outside air which enters through the access tunnel, circulates through the valve chamber and the power chamber and is exhausted through the inclined cable tunnel. The two main axial flow type exhaust fans, each of 30,000 cfm capacity, are located at the top end of the cable and ventilation tunnel, inside the administration building. The flow of air entering the tunnel portal is divided into two streams halfway down the tunnel; about 7,650 cfm is drawn into the valve chamber while the bulk of the air continues down to the power chamber. A fan mounted in the wall of the valve chamber exhausts the air into a vertical 5 ft diameter shaft leading down to the space between the generator hall drip roof and the concrete roof arch.

The remaining 51,150 cfm of fresh air is divided after entering the generator hall, into two portions: approximately 7,150 cfm is circulated through the basement and then returned to the cable tunnel, while the remaining 44,000 cfm flows across the generator hall, gradually picking up heat from the electrical equipment and rising to the ceiling. A number of adjustable blade dampers admit the air through the drip ceiling to the space above. The air continues then toward the access end of the power chamber where it is drawn into the top of a vertical shaft of 81 sq ft area. The lower end of the shaft discharges the warm air into the cable and ventilation tunnel from which the air is finally exhausted into the outside atmosphere by the fans in the administration building.

Additional smaller fans provide positive ventilating for the battery room, station service room, and washrooms. The control room is equipped with a 5-ton air conditioning unit for the comfort of the station operators and for the purpose of providing fairly constant temperatures and humidity, necessary for proper functioning of the instruments on the control panels.

No special provisions were made for the ventilation of the surge chamber, where some air circulation results from the connection to the valve chamber. Maintenance work inside the surge chamber will be scheduled during the low water season when additional ventilation is provided by the tail tunnel not flowing full.

It has not been necessary to dehumify the incoming air, as there is no objectionable condensation of moisture from the air along the rock walls of the underground chambers which are relatively warm at this geographical location. However, all electrical control switchgear cabinets have small electric heaters to prevent moisture condensation inside.

The design of the station drainage in an underground installation has to be quite liberal in view of the uncertainties involved in predicting the amount of seepage through the rock surfaces of caverns and tunnels. The valve chamber seepage is drained by gravity to the surge chamber and the tailwater. A good portion of the access tunnel also drains by gravity to the tailwater at a point, where the tailrace tunnel crosses underneath the lower loop of the access tunnel. Seepage collecting in the bottom end section of the tunnel and in the cable tunnel, is drained into a special sump placed at the entrance to the power chamber. The sump is equipped with two 500 gpm pumps operated in duplex and on automatic alternator control.

The power chamber drainage system consists essentially of a concrete floor gutter extending around the periphery of the cavern, drip roof downspouts

and a supplementing network of embedded pipe, connecting to a large sump located between units 1 and 2. The sump, which also serves for unwatering of the units, collects all seepage, turbine seal, and spiral case leakage, cooling water from the air conditioning unit, and compressors, and drainage from maintenance or cleaning operations. The cooling water from the generator, generator bearing, governor oil and turbine coolers is discharged directly to tailwater at each unit.

Four identical drainage pumps have been provided in order to simplify maintenance and reduce the number of spare parts to be kept in stock. The pumps are so sized that any one of them alone is suitable and adequate to handle normal inflow into the sump. Each pump is capable of pumping 1360 gpm against a static head of 35.5 feet which corresponds to the normal operating tailwater. At higher tailwater levels more pumps may be used to handle the inflow. The unwatering of a turbine requires all four pumps operating simultaneously.

Administration Building

The administration building erected next to the switchyard, is a small reinforced concrete structure, housing the plant superintendent office, the radio communication room, and auxiliary equipment such as: power station main exhaust fans and its emergency diesel driven 312.5 kva generator. A transformer maintenance shop adjoins the building.

The emergency diesel generator in addition to providing the power necessary for operating the power station auxiliaries, also furnishes power required in an emergency for raising or lowering the spillway gates. The gates are operated normally by remote control from the power station underground control room.

Project Ownership, Engineering, and Construction

The Ambuklao Project is owned and operated by the National Power Corporation of the Philippines. It has been financed by the Government of the Republic of the Philippines and by a loan from the Export-Import Bank in Washington, D. C. for the purchase of goods and services outside of the Philippines.

The Harza Engineering Company assisted the National Power Corporation in the study of the site and in the preparation of the report on the feasibility of the development. In the subsequent stage of design work Harza Engineering Company supplied design drawings, specifications, and finally, technical assistance to the engineering staff of the National Power Corporation in the preparation of construction drawings and in the supervision of construction. Field changes in the designs were carried out by the National Power Corporation engineers. Mr. Irving B. Crosby served as a consultant geologist for the development.

Principal contractors on the power facilities of the project were:

General construction	-	Philippine Engineers' Syndicate Inc.
Turbine and Governors	-	Neyrpic, France
Generators	-	International General Electric Company
Cranes	-	Ceretti and Tanfani, Italy
Switchgear	-	Westinghouse Electric International Co.

Transformers	-	International General Electric Company and Westinghouse Electric International Company
Penstock Valves	-	Charmilles, Switzerland

The tailrace tunnel was started by the Agvid Construction Company, Philippines, and completed with the help of the general contractor for the power facilities. The construction of the power tunnel and the diversion tunnel was started by the National Power Corporation, with its own forces, and later completed by the general contractor for the dam and the spillway, Guy F. Atkinson, USA. The first generating unit was placed on the line in December, 1956.

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INSURANCE ASPECTS OF NUCLEAR ENERGY[a]

Edward R. Lloyd,[1] J.M. ASCE
(Proc. Paper 1599)

SYNOPSIS

The intent of this paper is to give a better insight into nuclear energy insurance. Leading up to this goal, it will first be necessary to briefly discuss the fire and casualty insurance industry. The syndicates or insurance pools that have been formed to write insurance on nuclear facilities are discussed in some detail. The basic factors of the insurance rating plans for nuclear risks are explained. The approximate amounts of nuclear energy insurance available to industry from both private and federal sources are outlined.

INTRODUCTION

A logical beginning for this topic is with a definition of insurance. One definition frequently used is: Insurance is a means by which an existing burden of risk can be transferred from its bearer to others more able to bear it. The word "risk" as used in this definition means the uncertainty which exists regarding the occurrence of future events which threaten loss of one kind or another. By assuming this burden of risk, the fire and casualty insurance industry in this country has grown during the past 200 years into an organization where the annual premium income of the ten largest companies alone approximates three and one half billion dollars.

Growth of Insurance Industry

The story of the industrial growth of our country is also the story of the

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a. Presented February 27, 1958, before the Power Division, ASCE, Chicago Convention.

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development of the business of assuming risks which constantly increased both in variety and seriousness. The first fire insurance company organized in America upon a stable basis came into existence in 1752. In the ensuing hundred odd years many fire insurance companies began operation, some of which are still in existence today. Shortly after the organization of the National Board of Fire Underwriters in 1866 reasonable order and progress developed in the fire insurance business.

Casualty insurance was first written in the United States in 1850 against the perils of railway accident. Next in 1866 came steam boiler insurance. Then in 1867 came plate glass, 1885 burglary, 1886 employers and contractors liability and so on until 1898 when automobile liability insurance came into being and 1910 when workmen's compensation coverage began.

When one reflects back to the growth of our country and reviews the progress made in the various fields of engineering and technology, we can more readily understand the concurrent development of the insurance industry. This never ending growth of our civilization continues to require the insurance industry to keep pace. As new machines and processes are developed new perils and exposures appear against which indemnification is required. The insurance industry has kept pace through the advent of the railroad, the steam boiler, the automobile and on up through the present day giant and complex industrial plants. From a peril standpoint they have kept pace by affording indemnity against earthquake, fire, lightning, explosion, windstorm, hail, hurricane, mechanical breakdown and a host of other exposures.

And now, the peaceful uses of nuclear energy bring a new challenge to the insurance industry. Indemnity is required against both physical damage to nuclear facilities and against third party liability resulting from radiation or radioactive fall-out caused by an incident to a nuclear facility.

Insurance Syndicates

Insurance syndicates, or as they are sometimes called "pools", have been in existence in this country since before the turn of the century. Although formed for the purpose of insuring various specific types or classes of exposures, these syndicates objectives are all similar. Basically they are organized to furnish greater insurance capacity with expert service and uniform practice on a far more economical and efficient basis than could be achieved by each participating company acting alone.

Among the long established syndicates are (a) the Factory Insurance Association organized in 1890 and currently with approximately one hundred active member companies, (b) the Cotton Insurance Association organized in 1905 and currently with about thirty members, (c) the Oil Insurance Association established in 1918 with an active membership today of about 90 companies, and (d) the Railroad Insurance Association organized in 1920 after the return of the roads to private operation following World War I. This syndicate currently has about twenty active members. These are but a few of the insurance pools in operation.

The inherent hazards and potential loss possibilities involved with the use of nuclear energy have suggested to the insurance industry that syndicates be established for the purpose of indemnifying these new exposures. The development of other types of hazardous industries has had a relatively slow progression. This has permitted the insurance industry to gradually provide

the limits of liability and amounts of insurance required. In so doing, experience was gained and loss statistics were accumulated over a period of years. Insurance syndicates could, therefore, be formed as the need became apparent and with the benefit of the quantitative data of member companies. This gradual growth and the accumulation of data has not prevailed in the field of nuclear energy.

Nuclear Energy Insurance Program Organized

The Atomic Energy Act of 1954 provided private industry with the opportunity of engaging in non-military activities utilizing atomic energy. Private organizations, in particular the public utility companies, have shown a remarkable willingness to make available large sums of money and to place a high priority on nuclear energy projects. They have been encouraged by the vital interest the Federal Government has shown in the acceleration of the development of atomic power.

The insurance industry has been faced with the job of organizing a nuclear energy insurance program based in large on judgment factors and no past experience. The amount of maximum liability that may be required of a large reactor operator has been estimated in the testimony given before the Joint Congressional Committee on Atomic Energy at sums ranging from several hundred thousand dollars to sums up to a billion dollars or more. This wide range of estimates would tend to place the amount of maximum liability necessary in the incalculable category.

At the same time industry was designing uses for atomic energy, insurance companies were organizing management and engineering committees to study the problems of nuclear energy insurance. It was advantageous for some insurance companies to obtain Atomic Energy Commission Access Permits and clearances thereunder for some of their engineers and key personnel. In addition to this, some insurance companies have seen fit to make available educational opportunities in order that otherwise qualified personnel could receive a substantial background in nuclear energy.

During March, 1955 the AEC appointed an Insurance Study Group. This group is made up of ten men, each of whom is a top executive. They represent both capital stock and mutual fire and casualty insurance companies. The purpose of this group is to study the insurance problems raised by the introduction of nuclear energy and to develop information and criteria with respect to the insurability of industrial atomic energy installations. In addition, they advise and make appropriate recommendations to the AEC. An interim report was released in July, 1955 by this Study Group.

The men of the Insurance Study Group have, for the past two years or so, been guiding various committees. These committees, and in some cases subcommittees thereof, have been actively engaged in the study and development of a multitude of problems and solutions. Some are studying our present standard insurance policies and endorsements to try and ascertain how the use of atomic energy will effect the coverage afforded thereunder. Some are working on the wording of the forms, policies, and endorsements for excluding the nuclear hazard where this exposure is not contemplated. At the same time, others are developing policies for including the perils of nuclear reaction, nuclear radiation and radioactive contamination. Still other committees are engaged in the task of insurance rate development for this new

and unique class of business. The engineering personnel of various insurance companies are, through committees, organizing the necessary inspection facilities.

Concurrent with much of the above mentioned activity, the insurance capacity or market for nuclear energy coverages has been getting priority treatment. Both the capital stock and the mutual fire and casualty insurance companies began the organization of separate but similar associations to write physical damage insurance and third party liability insurance for nuclear energy users. This paper will touch on the "Mutual Pools" but will deal chiefly with the associations formed by the capital stock insurance companies. They are similar enough that the discussion of one outlines the other.

Nuclear Energy Insurance Syndicates

The efforts of all of the committees and negotiations have not been in vain. They have, in fact, produced three insurance pools or syndicates to write nuclear energy coverages. They are as follows:

1. Nuclear Energy Property Insurance Association (NEPIA)
2. Nuclear Energy Liability Insurance Association (NELIA)
3. Mutual Atomic Energy Reinsurance Pool (MAERP)

Taking these in the order listed, the first to be discussed will be the Nuclear Energy Property Insurance Association, referred to in the now common alphabetical form as NEPIA.

The capital stock insurance companies had in the National Board of Fire Underwriters their largest service organization. It was only natural, therefore, to set up the initial ground work for a nuclear insurance pool through this organization. A Special Committee on Industrial Uses of Atomic Energy was established by the National Board. Through this committee, a plan was formulated and submitted in December, 1955 to 366 capital stock property insurance companies. This submission was in the form of an invitation to participate in a syndicate or association to provide physical damage insurance for nuclear energy reactor installations.

Things moved ahead at a rapid pace and on May 25, 1956 an organizational meeting was held at which time the Nuclear Energy Property Insurance Association, a voluntary, non-profit, unincorporated association of insurers was formed. The original participants numbered 188 capital stock insurance companies who had made commitments from the minimum of \$ 25,000 upward to as much as \$ 4,000,000 each. Several additional companies have joined NEPIA since its origin.

The routine day-to-day operation of NEPIA is conducted through the long organized and well established facilities of the Factory Insurance Association. The control of NEPIA, however, is vested in a Governing Committee made up of nine member companies and assisted by an Advisory Committee of six.

The following types of risks have been set down as coming within the scope of coverage being written by NEPIA:

A. Nuclear reactor powered electric generating stations. This includes all auxiliary property on the premises.

B. Other nuclear reactor installations. This includes non-power type reactors such as those used for research, experimental, test or demonstration purposes. Any "Hot lab" in connection with such a reactor

installation may be included and "critical" facilities are also eligible.

C. Nuclear fuel element plants. This includes fuel element fabricating plants, fuel processing or reprocessing plants and fuel salvage or recovery facilities.

D. Nuclear fuel in transit. A special form of contract will be written covering nuclear fuel in transit to and from, but not while on, the premises of the several above mentioned types of nuclear facilities.

E. Nuclear risks while in course of construction. In order to be in a position to provide loss prevention and engineering services from the start of construction, NEPIA plans to write a special form of Builder's Risk insurance during the plant construction period.

The property damage insurance policy issued by NEPIA is entirely new in form. The main object has been to include indemnification, within reason, against the perils of nuclear reaction, nuclear radiation and radioactive contamination. To produce this policy has brought about many interesting and heretofore unencountered situations. In the event of a reactor incident, it may be next to impossible to establish the actual cause of the loss. It could be fire, explosion, mechanical or electrical breakdown, nuclear reaction, etc. or a combination or overlapping of any of these. It was, therefore, desirable to so construct a policy that it would be all-risk in character. If this had just been a matter of combining the common fire and extended coverages it would have been less cumbersome. But the requisite of including a portion of the boiler and machinery coverage, traditionally written by the casualty companies, added to the difficulties.

Boiler and machinery insurance affords coverage against physical damage to insured objects as well as property damage and bodily injury liability. It has been found advantageous in the case of nuclear energy insurance to split these coverages, the pure physical damage portion being assigned to NEPIA and the liability coverages being assumed by the casualty syndicates. Relatively few insurance companies are staffed with licensed boiler and pressure vessel inspectors. Therefore, those members of NEPIA who have boiler and machinery insurance and inspection facilities have, by agreement, been assigned that phase of the NEPIA insurance program.

As for insurance capacity, NEPIA currently has a gross underwriting capacity of approximately 58-1/2 million dollars per risk. Additional amounts are still being subscribed by both the domestic and foreign markets and it is anticipated that the capacity will soon reach $60,000,000 per risk.

The second syndicate to be discussed is the Nuclear Energy Liability Insurance Association, referred to alphabetically as NELIA. This insurance pool was organized on May 9, 1956 to provide third party liability insurance on privately operated nuclear energy facilities.

An invitation to participate in NELIA was extended to all capital stock casualty and surety insurance companies, as well as to fire insurance companies having casualty facilities. The Association currently has approximately 140 members.

The routine management of NELIA is conducted through the facilities of the Association of Casualty and Surety Companies. The control of NELIA, however, is vested in a Governing Committee made up of nine member companies. Several subcommittees of the Governing Committee have been appointed for specific duties. In addition, several standing committees were duly appointed. Among these are:

1. The Accounting and Statistical Committee
2. The Committee on Underwriting
3. The Committee on Claims
4. The Committee on Engineering and Inspection

Third party liability insurance is available through NELIA for such installations as reactors, critical facilities, fuel fabricators and to users, handlers, processors or reprocessors of fuels. Because of the numerous factors involved it has, at this stage of progress, been found desirable to treat each facility specifically. In evaluating an exposure the following factors are given consideration:

1. The type of facility;
2. The use of which it is to be put;
3. The power level;
4. The containment;
5. The location (that is, the proximity to populated areas and property values).

The bodily injury and property damage liability insurance policy being issued by NELIA agrees to pay all sums, within the limits of the policy and subject to the exclusions and conditions therein, which the insured shall become legally obligated to pay as damages because of:

1. Bodily injury, sickness or disease, including death resulting therefrom, sustained by any person;
2. Physical injury to or destruction or radioactive contamination of property, and loss of use of property so insured, destroyed or contaminated, and loss of use of property while evacuated by order of public authority because of possible contamination;

caused by the nuclear energy hazard. The term "nuclear energy hazard" is defined as the radioactive, toxic, explosive or other hazardous properties of source material, special nuclear material or by-product material as respects the coverage being afforded.

NELIA and its Mutual counterpart have joined together and produced policies that provide similar coverage. They have set up uniform standards for underwriting, engineering, inspection and claims handling. In addition, they have established a combined rating system.

The original capacity sought by NELIA was fifty million dollars per risk. The members of NELIA, stock casualty insurance companies, exceeded their own expectations and currently have a gross underwriting capacity in excess of $50,000,000.

The third and last syndicate coming under this review is the Mutual Atomic Energy Reinsurance Pool, called MAERP for convenience. MAERP is not an entity but merely an agreement between a group of Mutual insurance companies. It is a combined physical damage and public liability organization whose activities encompass those of both NEPIA and NELIA. This insurance pool was organized on April 6, 1956, and all mutual fire and casualty insurance companies whose underwriting powers would enable them to do so were invited to participate.

The long established Senior Factory Mutuals act as the policy issuing organization for the physical damage insurance coverage written through MAERP. The Mutual Atomic Energy Liability Underwriters (MAELU) was

formed to operate as a policy issuing organization for public liability coverage.

The gross insurance capacity of MAERP totals approximately $25,000,000. The physical damage portion of this total is about 10-1/2 million dollars while the public liability share is roughly 14-1/2 million dollars.

The insurance policies, forms, general operating procedures, etc. of the Mutuals are concurrent with those of NEPIA and NELIA. In fact, many joint committees made up of representatives from both the Stock and Mutual organizations have worked and are working together with common objectives and goals. An example of this cooperation is the inspection team arrangement for public liability coverage. These teams are made up of a member from NELIA and one from MAERP who report their findings to a Special Joint Subcommittee of the Engineering Committees of NELIA and MAERP.

Nuclear Insurance Rating

The combined efforts of the Stock and Mutual insurance syndicates were also put to task on the problems of insurance rating for nuclear energy risks. To date there is no factual or actuarial data upon which an estimate of the frequency of expected losses or their possible magnitude can be made. This, together with the absence of standardization in design and type of installation, makes the rating of nuclear risks on an individual risk rating basis desirable.

A new insurance rating organization, Nuclear Insurance Rating Bureau (NIRB), was organized in January of 1957 to formulate the rates for nuclear energy property damage insurance. This Bureau has a membership of approximately 300 Stock and Mutual Insurance companies.

Under the rating plan that has been drawn up by NIRB, the full basic fire, extended coverage and vandalism rates are used as a starting point. To this is added the boiler and machinery rate for the objects in that category. Then a rate charge for the nuclear perils is added. Finally an increment for the all-risk exposure not covered by any other rate charge is figured in.

To determine the charge for the "nuclear peril" mentioned above two rating guides have been developed. One evaluates the hazard for plants handling radioactive materials and the other evaluates the hazard of reactor and critical facilities. For those plants handling radioactive materials the rating plan gives consideration to three basic conditions. They are:

1. A basic Nuclear Hazard Factor which establishes a charge for the kind of nuclear material present in the process; (natural or enriched uranium, plutonium, etc.)
2. A Secondary Rating Factor or charge for materials, other than nuclear, such as pyrophoric metals;
3. A second Secondary Rating Factor evaluating the process involved, such as chemical processing, cladding, machining etc.

The rating plan for reactors and critical facilities gives consideration to the following:

1. Type of Reactor; (PWR, Swimming Pool, etc.)
2. Use; (Power, Testing, Medical, Industrial or Educational Research)
3. Power Level; (Megawatts of heat)
4. Containment.

From the above, a final "Valve Unit" is produced for any given installation. This is converted to a monetary increment to be included in the composite rate.

The rating of nuclear energy liability insurance is being handled by a combined Stock and Mutual rating committee. NELIA is represented by the Nuclear Energy Liability Division of the National Bureau of Casualty Underwriters. MAELU is represented by a Sub-Committee of the Mutual Insurance Rating Bureau.

The liability insurance rating plan for reactor facilities is very similar to the procedure outlined above for physical damage insurance. It contains provision for the consideration of type, use, power level, and containment. To these is added a factor for the location of the reactor in relation to surrounding human and property exposures. A base premium is established for the first million dollars of insurance. Thereafter diminishing percentages of the base premium are used for several established increments up to $50,000,000 of insurance.

For critical facilities and fuel fabricators fixed rates for the first million dollars of insurance and for each additional million have been set up. In addition, rates have been established for other types of coverage, including coverage for designers of reactors, and for Stand-by coverage. The latter is effective only up to the time a radiation exposure exists at an insured location, such as during the construction period.

The rating systems that have been developed are by necessity based on underwriting judgment together with all available current information. The liability pools have made provision whereby a retroactive downward adjustment of rates may be made after a period of years if substantial losses do not occur.

Nuclear Insurance Capacity and Limitations

In the above discussion, the gross insurance capacity of each syndicate was mentioned. To this must be added the provisions made available by the amendment dated September 2, 1957 to the Atomic Energy Act of 1954. This amendment reads in part:

> "In order to protect the public and to encourage the development of the atomic energy industry, in the interest of the general welfare and of the common defense and security, the United States may make funds available for a portion of the damages suffered by the public from nuclear incidents, and may limit the liability of those persons liable for such losses."

The law states that a condition of each license issued by the AEC shall be a requirement that the licensee have and maintain financial protection of such type and in such amounts as the Commission shall require to cover public liability claims.

The amendment in question makes available, for a fee, an aggregate indemnity of $500,000,000 for all persons indemnified in connection with each nuclear incident. The law also limits the aggregate liability for a single nuclear incident to a sum not exceeding $500,000,000.

To supplement the underwriting capacity of the domestic market, the

foreign reinsurance market has subscribed an amount in excess of $43,000,000. This is divided among NEPIA, NELIA and MAERP on a quota share basis. The amounts allocated to each are included in the gross capacities already mentioned. It is interesting to note that the foreign reinsurance is being supplied by sources in approximately twenty countries throughout the world in amounts from $10,000 to almost $15,000,000.

CONCLUSION

From the foregoing it is hoped the conclusion can be reached that the insurance industry has met the nuclear energy challenge. The nuclear energy insurance syndicates have a number of risks insured under binder and the formal policy contracts should be ready in the very near future. There will undoubtedly be a trial and error period of adjustments and policy changes. This can be expected of anything as large and unique and with as many unknowns as nuclear energy insurance. But let it suffice to say that the nuclear insurance syndicates, NEPIA, NELIA and MAERP are in business.

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CIVIL ENGINEERING ASPECTS OF THE DRESDEN NUCLEAR POWER STATION[a]

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(Proc. Paper 1600)

ABSTRACT

A comprehensive description of arrangement and Civil Engineering design features of the Dresden Nuclear Power Station is presented. Design consideration arising from the layout of nuclear power equipment within a vapor tight sphere are discussed. Civil Engineering problems of more conventional nature are reviewed and related to nuclear power plant design.

INTRODUCTION

Civil Engineering has been described as the art and science of developing the great resources of nature for the benefit of man. Certainly, the development of systems for the generation of electrical power from the nuclear fission process is one of the most significant forward steps made by man. The civil engineer in company with his fellow engineers and scientists has been a partner in this development.

This paper emphasizes the Civil Engineering aspects of the Dresden Nuclear Power Station, a 180,000 kilowatt all nuclear plant being designed and built by the General Electric Company. Primary emphasis is on the arrangement, coordination, and interrelation of plant components in a workable nuclear power plant design. The nuclear, mechanical and electrical systems

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a. Presented at a meeting of the ASCE, Chicago, Ill., February, 1958.

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are covered briefly.*

The plant is located about 50 miles southwest of Chicago at the confluenece of the DesPlaines and Kankakee Rivers. Financing is being provided solely by private industry. General Electric is building this nuclear power plant for the Commonwealth Edison Company and members of the Nuclear Power Group for $45 million. This group is composed of: American Gas and Electric, Bechtel Corporation, Central Illinois Light Company, Commonwealth Edison Company, Illinois Power Company, Kansas City Power and Light Company, Pacific Gas and Electric Company, and Union Electric Company of Missouri. The Bechtel Corporation is the engineer-constructor for the plant under contract with the General Electric Company.

The Dresden Station is to be completed in 1960. Design started in April of 1956 and major construction started early in 1957.

An artist's conception of the completed facility is shown in Figure 1, illustrating the compact nature of the arrangement. This view is taken looking north. The reactor vessel and its associated equipment are housed in the 190 foot diameter steel sphere shown on the right. The large rectangular building to the left of the sphere contains conventional power plant components, such as the turbine-generator, condenser, feedwater heaters, and control room. An Administration Building and Access Control Building are located in front of the Turbine Building. Shop-Warehouse and Fuel Storage Buildings are shown in front of the sphere. A Radioactive Waste Facility is seen on the extreme right. Circulating water to the main condenser in the Turbine Building enters via the canal on the right and is discharged by the canal on the upper left.

The Site

The site for the Dresden Station is roughly one mile wide by one and a half mile long. As shown in Figure 2 the site is located at the juncture of the DesPlaines and Kankakee Rivers where the Illinois River is formed. The plant itself is located at least one-half mile from any off-site land as a safety feature. This location on the site, however, necessitated long canals for circulating water, as can be seen in the figure.

The ground surface at the site is gently rolling, and the site is easily drained. However, bed rock underlies the general plant area within four feet of the ground surface. Construction requirements at the site have required the excavation of approximately 150,000 cubic yards of rock.

The site is served by the Elgin, Joliet and Eastern Railroad. A spur has been built from the railroad at the site western boundary to the plant area. Highway access is from the south. The Illinois waterway will be used for barge access to the site.

The switchyard will be located to the north of the plant. Commonwealth Edison Company is building the outgoing transmission lines.

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*A more detailed description of the Dresden Station may be found in the General Electric publication GER-1301, "A Design Description of the Dresden Nuclear Power Station." (ASME paper No. 56A169, Nov. 26, 1956).

Unique Factors in a Nuclear Power Plant

Any modern complex industrial plant presents problems of arrangement, structural type, and economy which are strongly influenced by the process for which the plant is built. This is certainly the case with a conventional fossil fuel fired power plant. The problems to be briefly reviewed here are those which have been super-imposed on the power plant design by virtue of its being a nuclear plant.

These factors have a strong influence on plant arrangement. However, these are not problems beyond the scope of our knowledge. Application of judgement, common sense, and more than the usual care will solve most or all of the new problems associated with nuclear power plant design.

Radiation and Shielding

The unique factors in a nuclear plant, which have the strongest influence on design and arrangement, stem directly from the fact that the nuclear process produces ionizing particles and rays. Under certain conditions this radiation can be biologically harmful to man. Therefore, shielding protection from neutrons and gamma rays is required in certain areas.

The neutrons are generated in the core of the reactor and present a radiation problem in the immediate area around the reactor core. The thickness of the shielding concrete opposite the reactor core has been dictated by the neutron radiation from the core.

Water passing through the reactor is irradiated in the dense radiation flux in the core. The general result is that certain isotopes are created by fast neutron reactions in the water. Of those isotopes created, nitrogen-16 is generally most important in its effect on shielding design.

The 6 mev gamma rays originating from the decay of nitrogen-16 control the amount of shielding required for pipes and equipment carrying reactor water and steam. In some locations, corrosion products and/or ruptured fuel particles will dominate, for example, in the demineralizers and cleanup equipment.

Fortunately, the nitrogen-16 has a short half-life so that equipment which may contain reactor water or steam will generally be accessible within a few minutes after plant shut-down. However, radiation from corrosion products may still limit access in some areas.

Operation of the plant requires that the radiation be confined to the immediate vicinity of the equipment which is the radiation source. This is the reason for the relatively thick and massive concrete construction which is so characteristic of nuclear plants. Gamma radiation can be stopped or shielded most effectively by mass. Experience indicates that ordinary portland cement concrete is one of the most versatile and economical materials which can be used for shielding.

Heavy aggregate concrete, steel, and lead have been used in some nuclear plants for shielding where space was at a premium. One of the design objectives in the arrangement of the Dresden plant was to keep the use of such materials to the bare minimum for shielding. That the designers have succeeded in this objective is indicated by the fact that less than twenty cubic yards of heavy aggregate concrete have been called for in the design.

Directly related to the problem of radiation and shielding is the fact that much of the plant system equipment is not accessible for adjustment or maintenance during operation. Instrumentation and control features will permit the station to operate with a degree of reliability and availability comparable to a modern conventional plant. Provisions for isolation of certain equipment, such as feed-water heaters, will permit direct maintenance without shutting the plant down.

Provision for Decay Heat

In a conventional plant when the fire is out the boiler will cool down. It can then be physically inspected and repairs made directly as required. A nuclear reactor, of course, can be shut down. However, even after shut-down the reactor core continues to give off considerable heat. This decay heat, comes from the gradual nuclear decay of the fission products which have been formed in the reactor during operation. Since it is not economically desirable to remove all the fuel from the core at every shutdown, this heat must be dissipated. In the Dresden Station a shutdown heat exchange system will remove the decay heat.

Radioactive Wastes

The operation of this plant will not produce any flyash but it will produce some waste products which must be disposed of. First of all the ventilating air which is circulated through the process buildings will be discharged to the atmosphere through the 300 foot high stack. There is a remote possibility that this air may pick up some contamination on rare occasions and therefore, it will be routinely monitored before being discharged through the stack. Also, the air which passes the outside of the reactor vessel will be irradiated by those neutrons which escape the vessel. This air must be discharged at a safe location, in this case from the stack. The air ejectors in the condenser system will remove air in-leakage and gases formed in the reactor and turbine systems. These gases will be sent up the stack after suitable delay to permit decay of the nitrogen-16. As mentioned previously, all gases discharged from the stack will be monitored to insure environmental protection.

In general, liquid waste disposal will be to the Illinois River. A comprehensive radioactive waste disposal plant is provided for processing such waste material which must leave the plant. Disposal will be constantly checked and controlled. Concentration and on-site storage of the more radioactive liquid waste is also provided for. Solid waste, whether it is broken glassware and filter papers from the laboratories or highly contaminated equipment which is no longer serviceable, will be disposed of in concrete underground vaults on the site.

While beyond the scope of this discussion, it can be stated that all waste disposal from the plant will be carefully regulated and held within the maximum permissible rates set by the regulatory groups.

Provision for Access Control

One feature of this plant which has had an influence on total plant layout is the need to control the entry and exit of operations personnel in the areas

which are potential radiation zones. It was recognized early in arrangement studies that this control over personnel was a feature which would have to be built into the plant rather than leaving it as an operational problem to be solved later.

The Dresden Station, therefore, includes an access control building. Figure 3 shows the relationship of the Access Control Building to the total plant layout. The shaded areas in the figure are those which are subject to control from the Access Control Building.

Operating personnel desiring to enter any of these areas must pass the control point in the Access Control Building. At this point the required authorizations will be checked and each man will acquire the necessary equipment for his own protection.

When a man returns from his duties in the controlled area he again passes the control point where he is cleared for access to the rest of the plant.

A central location was chosen for this building. It is physically close to the turbine building, the reactor enclosure, the shop, and the fuel handling building. Access to all or parts of these buildings must be controlled. And further, no cross connections could be permitted. That is, the controlled area could not have uncontrolled corridors, for example, cutting across it. While the final result as shown in the figure looks simple and logical, a great deal of effort went into the planning of this facility before arriving at a satisfactory arrangement.

Nuclear Steam Supply System

Plant Cycle

Figure 4 shows a simplified schematic of the power cycle selected by General Electric to fulfill the design requirements of the Dresden Station. Light water, serving both as coolant and moderator, passes through the reactor core of slightly enriched uranium dioxide and forms a steam-water mixture which is routed to a "primary" separating drum. The steam from this drum flows directly to the turbine at 965 psig and 541°F. Water from the drum is returned to the reactor through four "secondary" steam generators. Feedwater to these generators removes additional energy from the recirculating water to form "secondary" steam for admission to the turbine at 500 psig and 461°F. The term "dual cycle" is derived from the use of these two sources of steam.

Reactor Package

The essential equipment of the nuclear steam supply system for the Dresden plant consists of the reactor vessel, the primary steam drum, emergency condenser, secondary steam generator, recirculating pumps and associated high pressure piping system. The general arrangement is shown in Figure 5.

This equipment is housed in a 190 foot diameter steel sphere designed to contain fission products in the remote chance of a serious accident. Special features such as escape locks, and leak tight openings for mechanical and electrical penetrations are provided in the design of the enclosure to satisfy the plant arrangements. The sphere extends thirty-nine feet below grade

forming a suitable base for the equipment and biological shielding associated with the nuclear steam supply system. Directly below the reactor vessel, vertical space is provided for the removal of the bottom-mounted control rods. Surrounding this area are rooms for control system piping and equipment, and for auxiliary equipment. The reactor vessel plus the refueling area above the reactor accounts for approximately 61 feet in height. The height of the steam drum above the exit elevation of the riser pipes is dictated by NPSH requirements on the recirculating pumps together with other hydraulic considerations. The emergency condenser, which forms an automatic heat sink for certain reactor shutdown conditions is located 23 feet above the center line of steam drum. In a horizontal plane, the limits of the reactor package are based on the space required for the secondary steam generators plus the shielding walls in one direction and the refueling area in the other.

Occupancy During Operation

To achieve the necessary flexibility of operation during both normal plant operation and required down time, provision for access to key equipment has been incorporated in the arrangement of the plant. For example, the secondary steam generation equipment is divided into four separate loops contained in separate shielded compartments. Maintenance of equipment in any loop during plant operation is possible with the equipment shut down. Certain equipment is provided in duplicate with shielding walls arranged to permit access for maintenance during operation.

The overall influence of providing habitable areas during operation requires that the arrangement of the package be spread and expanded beyond the minimum needs of shielding the equipment only. Briefly, the access spaces are shielded for radiation levels consistent with operational and maintenance requirements.

Reactor Vessel

The reactor vessel, 12 feet 2 inches inside diameter, 41 feet high, with a 5-5/8 inch wall, weights approximately 580 tons in an operating condition and 820 tons during the refueling cycle. The type of support for the reactor vessel was selected to best satisfy static loading, space limitations and position in elevation for anchoring or being the "zero" point for the complex piping system connected to it. Twenty-four supporting brackets are built into the perimeter of the cylindrical portion of the vessel and spaced as close to the 22 inch recirculating return nozzles as possible. These brackets are supported on steel bases which transfer the vessel loads into the reinforced concrete foundation. Radial movement of the vessel from thermal expansion is taken on Lubrite plates at the contact surfaces between the vessel brackets and ring girder. Four guides at the quarter points are provided 28 feet above the support plane to maintain vertical alignment when the vessel expands from thermal growth. These guides allow vertical and radial movement from thermal expansion while keeping the vessel in vertical alignment. A design load of .066 G based on the operating weight of the reactor vessel takes care of seismic loading.

The Dresden reactor uses bottom entry vertical control rods which require a space approximately 15 feet in diameter and 36 feet deep for the rod mechanisms, their associated piping and room for removal for maintenance.

The auxiliary equipment for the control rods is located in rooms adjacent to the control rod space.

The primary shield around the reactor vessel is comprised of 11 feet of ordinary reinforced concrete. Cooling coils within the primary shield are located within the inner 2 feet of thickness and will be used to keep concrete temperatures below 200°F. Where the primary shield is penetrated by large piping, space is provided for its installation and the opening closed after erection and test with concrete block. In areas requiring embedded sleeves, careful layout of the sleeves maintains the overall shield intergrity.

Primary Steam Drum

The primary steam drum is located 83 feet above the riser piping of the reactor vessel. This drum is approximately 65 feet long and weighs 204 tons in an operating condition. Twelve 16 inch risers from the reactor vessel together with ten 16 inch downcomer lines to the secondary steam generator piping loops are connected to the drum.

The steam drum is hung on twenty constant support hangers to minimize the length of piping to and from the drum and take care of the thermal expansion of the piping system without overstress. These carry the weight of the drum and part of the piping load. Each hanger is designed for a travel of approximately 6 inches under load. Limit stops are provided on each end of the drum. These are framed into the floor above and are designed to prevent tipping along the longitudinal axis of the drum during operation. Horizontal ties with adjustment are provided to take care of lateral and possible seismic loads.

Emergency Condenser

The emergency condenser is located with its center line 23 feet above the center line of the primary steam drum to provide the necessary elevation head to drain the condensate from the tube bundles. The tube bundles are located at each end of the tank shell. The entire weight of the unit is carried through support feet on the condenser tank shell.

Secondary Steam Generator Equipment

The Dresden plant has four secondary steam generators, recirculation pumps and the necessary piping loops. The secondary steam generators are anchored in place, with suitable provision for radial expansion of the shell. The recirculating pumps are supported by constant support hangers and float with the movement of the piping system. Provision is made for the complete removal of a recirculating pump and motor, less the fixed bowl, by lifting the assembly through hatches above.

Auxiliary Facilities

In addition to the equipment associated with the primary and secondary steam systems, space and shielding is provided for closely related auxiliary facilities. The shutdown heat exchangers and pumps used to dissipate residual heat during refueling are located as close to the reactor vessel as possible and still provide access for maintenance during operation. The dual system of demineralization equipment is heavily shielded and space provided

for maintenance. Cooling water equipment, air conditioning units and various shielded areas for special instrumentation are provided. The location and levels of piping penetrations for main piping as well as service water of various kinds are all factors determined by the civil engineering aspects of the overall plant.

Personnel and Equipment Access

Access to various operating levels within the sphere is still another aspect of civil work. Normal personnel access into the sphere is provided by the personnel access lock located 12 feet 6 inches above established grade. An operating floor is provided all around the main shielded structure at this level. Stairways at each end of the structure lead upwards to other operating levels and down to work areas below grade. Supplementing the stairways, a personnel elevator is provided to certain key operating levels for use during refueling and shutdown periods. An emergency escape lock is located diametrically opposite the personnel lock and at the same elevation.

To provide for the replacement of certain large pieces of equipment during the life of the plant, a 16 foot diameter bolted on cover is provided at an elevation approximately 12 feet above grade. This is located as close to the service of the 50 ton reactor crane as possible.

An equipment removal lock with doors having a clear opening of 8 feet by 8 feet is provided at an elevation 48 feet above grade. The passage through this lock leads directly to the turbine building where equipment can be handled by the turbine building crane. Within the sphere, the elevation of the equipment lock coincides with the operating floor over the secondary steam generator cells. Therefore, equipment can be moved directly from these cells to and out the lock as necessary.

Refueling

Water shielding is used in the Dresden plant to refuel the reactor. This means that the various functions of removing bolts, lifting heavy equipment, moving of fuel in and out of the vessel must be performed under water of various depths to give the necessary shielding for operating personnel. An open top concrete pool or canal, 26 feet in height above the lip of the reactor vessel is provided for this purpose. During normal operation, this canal is completely empty of water. When it is necessary to refuel the following sequence is followed. The canal is partially flooded and the reactor head is unbolted and stored in one end of the canal under water. Various removable fixtures within the vessel are also stored in the canal. With the proper depth of water in the canal the spent fuel is lifted out of the vessel, using special arms and grapples and placed in a rack carrier near the vessel. This carrier moves the fuel to the south end where it is placed in a basket sized to fit a 42 inch vertical pipe. The basket is lowered through this pipe to a receiving carrier which maintains a water seal between the canal inside the sphere and the storage area outside the sphere. A tunnel connects to storage and work areas described later.

To use this method of refueling, several items of interest to the civil engineer are noted. One is the seal of the reactor vessel to concrete required because the canal and the reactor vessel are flooded during refueling. This seal must provide for the expansion of the vessel both radially and

vertically from the cold position to the hot operating condition. A large stainless steel bellows seal integral with a metal skirt provides the seal and accommodates the movement. Another item of particular interest is the handling equipment which is provided for refueling. To assist in this operation a 50 ton crane with auxiliary hook, a special movable platform with hoist carriers, and an underwater rack carrier are provided.

Erection Sequence

A point of particular interest from a civil engineering aspect in the primary steam supply area is that of planning the erection sequence. The steel containment structure is being built first. This structure will be completed and tested for leakage and structural requirements. During this phase of construction and testing the shell will be entirely supported on steel columns.

At this stage a 24 foot diameter construction opening will be cut in the shell approximately at grade on the east side. Placement of foundation concrete inside and outside the sphere will be carried on simultaneously to bring the work up to approximately grade elevation. This will involve placing about 3000 cubic yards of concrete outside the sphere and about 11,000 cubic yards inside the sphere. Another construction opening will now be cut in the sphere at the north side. This opening will be approximately 22 feet wide by 29 feet high, located at grade elevation. In addition to serving as a construction access opening it will be large enough to permit entry of the reactor vessel.

The configuration of structures coupled with the size of the equipment is such that the support deck for the emergency condenser must now be erected starting from grade. The emergency condenser is shipped in sections of a size permitting field handling to its final elevation. After the emergency condenser is in place, the primary steam drum can be hoisted in a horizontal position to its final location just below the emergency condenser deck and supported by its spring hangers. The large piping connecting to the drum can then be brought in through the ends of the partially complete structure. The reactor vessel will then be brought into the sphere on special skids in a horizontal position. Sufficient structure will be erected to lift the reactor vessel in place. Work can then proceed to finish the additional shielding structures and complete the piping assemblies. The total concrete work associated with the reactor enclosure is about 29,000 cubic yards.

Fuel Building

The fuel building together with the storage areas for fuel is the facility where the Dresden fuel is stored temporarily and made ready for off site transportation. A vault for the new fuel is provided adjacent to the main structure. The fuel building structure above grade is 61 feet by 95 feet by 46 feet high from grade to top of roof. This portion above grade is of conventional design throughout. A seventy-five ton capacity crane, used for handling fuel casks, services the building. In addition, a special one-ton capacity crane equipped with a telescoping grapple device is provided for moving fuel rod assemblies under water.

The fuel transfer tunnel, handling pool, and storage pool below grade are of special interest. The fuel handling tunnel has a cross section 7 feet 9 inches by 15 feet 6 inches in height with its invert 42 feet below grade.

This tunnel extends 64 feet from the 42 inch diameter discharge tube below the reactor enclosure to the open topped fuel handling pool. The fuel carrier, mounted on tracks within the tunnel is used to convey the used fuel from the vertical discharge tube to the fuel building or conversely to take new fuel to the discharge tube. During operation, this tunnel and associated pools are filled with water of high purity. To prevent any possible leaks, either into the tunnel from ground water, or out from the tunnel or pools the outside surfaces are membrane waterproofed. Since part of the tunnel extends below the sphere and the rock excavation required extensive blasting, it was necessary to construct the structures below grade prior to erection of the sphere.

The fuel handling pool, 20 feet by 26 feet by 41 feet deep, provides a facility where the fuel can be removed from the carrier, inspected, and moved to other equipment or areas. For temporary storage, a pool 20 feet by 29 feet by 25 feet 5 inches deep is used. Fuel is moved under water through a movable bulkhead by using the special crane and grapple mentioned above. During normal operation, all the pools are flooded. By inserting the movable bulkhead between pools, it is possible to maintain the required depth of shielding water in the used fuel storage pit and dewater the tunnel and handling pool for maintenance and inspection.

Inside the fuel building and adjacent to these pools, a railroad well and siding is provided. Shipping casks for transporting used fuel from the plant site to reprocessing areas can be loaded here on railroad cars.

Turbine Building

General Arrangement

The turbine building houses the power generating and heat recovery systems of this plant. In addition, the control room, central control point for the plant, is located in the turbine building.

Figure 8 is a cutaway isometric of the turbine building. For reference the turbine floor is about 35 feet above grade. The turbine building has been so arranged that a logical separation of potentially contaminated and radiation source equipment is centrally grouped. This results in a functional arrangement as well for the major pieces of equipment. Grouped around this central area are areas containing equipment which are not radiation sources.

This division of equipment and the consequent division of the building into two general areas requires that there be access control. As has been discussed an access control building is provided for separation and protection of employees.

The logical arrangement of the turbine building demanded that convenient access to the controlled portion of the building be made. Corridors have been provided from which the radiation zones may be entered.

In general the radiation levels in the access corridors are sufficiently low that continuous occupancy can be tolerated. However, operation of the plant will not require continuous occupancy of these corridors.

For operating convenience access to the controlled corridors and controlled portion of the building can be made at points other than the normal entrance through the access control building. When these additional entrances are used, a local control station can be set up to make certain that contamination is not spread from the controlled areas.

The centerline of the turbine-generator coincides with an extended diameter of the reactor enclosure. This arrangement was chosen because it offered maximum simplicity and symmetry of the main steam piping.

Turbine Generator

The turbine generator is located approximately 35 feet above grade elevation. The turbine is an 1800 rpm tandem-compound, dual-admission machine rated at 192,000 kw when operating at 2.5 inches mercury exhaust pressure and one-half per cent makeup. The generator is rated 245,000 kva with 30 psig of hydrogen pressure and 0.85 power factor.

The turbine will be operated on saturated steam with no superheat. The fact that steam generated directly in the reactor will be fed to the high pressure section of the turbine will make the machine a radiation source during operation. Control of the turbine is arranged for remote operation.

Condenser

From a required shielding standpoint the condenser is rather ideally located. The major source of radiation, as has been discussed, is nitrogen-16. It was found during plant arrangement studies, that the condenser's location under the turbine and in practically the center of the building made it fairly simple to shield. The large volume of steam and condensate in the condenser will prevent access during operation. However, because of the short half-life of the nitrogen-16, the condenser should be accessible shortly after plant shutdown.

Circulating water for the condenser enters the turbine building from the intake structure through two 72 inch concrete lines approximately ten feet below grade. The circulating water is discharged to the discharge canal through a 96 inch concrete pipe. The intake and discharge structures are described briefly in this paper.

The condenser is a horizontal, single-pass, divided water box type. Retubing of the condenser will be done from the north side. To provide for this necessity and at the same time fulfill the shielding requirements, a concrete block wall will be erected opposite the condenser tube area. This wall can be taken down when retubing the condenser becomes necessary. Leaking condenser tubes can be plugged without removing this wall.

The three condensate pumps are located under the generator just to the west of the condenser. The pumps are located below the floor with only the drives projecting above the floor. It is anticipated that this area will be accessible during operation. A shielding wall is interposed between the condenser and the condensate pump room.

The gland seal condensers and the air ejectors are arranged along the north wall of the building in shielded rooms. This equipment is enclosed in two separate compartments each containing a condenser and an ejector. This duplication will permit one set of equipment to be maintained while the other set carries the normal plant operation.

Additional equipment along the north wall includes the turbine lubricating oil reservoirs, pumps and filter.

Feedwater Heaters

To the south of the turbine are located three compartments which contain the equipment for the five stages of feedwater heating. This equipment

handles radioactive steam and condensate and consequently shielding is required to protect the adjacent areas. In a conventional plant it is common practice to adjust and maintain this equipment during operation. In the Dresden plant operational adjustments to this equipment are made from an operating gallery just to the south of these feedwater heater cells. The gallery, of course, is shielded from the equipment.

The division of the equipment into three groups has been done to provide access for maintenance during operation. The equipment in any one cell can be completely isolated from the system and certain maintenance work can then be done. The radiation levels, even after shutdown of the equipment, may be higher than in the uncontrolled areas. Access, however, will be strictly controlled to insure that personnel are protected at all times.

This arrangement is an excellent example of the problems posed by radiation, and the manner in which they can be solved.

Feedwater Pumps

The feedwater pumps are arranged in two groups of three each. The secondary feed pumps supply feedwater to the secondary steam generators. The primary feed pumps supply feedwater to the main steam drum. Inasmuch as the primary feedwater has been demineralized the radiation potential will be very low. This accounts for the difference in shielding around the secondary pumps as compared to the primary pumps.

Control Room

The control room is located at the west end of the turbine building under the turbine room floor. This location was decided upon after much study. This room is the operating center of the plant: the point from which the reactor, turbine, and generation will be controlled. In the extremely unlikely event of an accident to the nuclear portion of the plant, it is essential that operating personnel be available to control the incident. The location of the control room provides maximum shielding protection to operators at a minimum cost.

The remainder of the turbine building at the west end and on the south side houses conventional equipment which is not a source of radiation. These areas are completely accessible for operation and maintenance purposes.

Supporting Facilities

Intake Structure

An intake structure to the north of the turbine building is provided. Two 90,000 gallons per minute pumps will take chlorinated and screened river water and circulate it through the condenser. This water is discharged through a discharge structure. The intake structure also houses three vertical turbine pumps, each with a capacity of 4300 gpm, used for the service water system and the fire system. An emergency diesel-powered fire pump is also housed in this structure.

Service Water

The service water system removes heat from hydrogen and lube oil for the turbine-generator, air conditioning equipment, fuel storage pool, waste and drain tanks, the reactor during shutdown, and two separate closed-loop cooling water systems, one in the turbine building and one in the reactor enclosure. The service water is discharged through an underground pipe to the discharge canal.

Cooling Water System

The two cooling water systems use demineralized well water with an anodic inhibitor added to minimize corrosion. The turbine building system removes heat from service and instrument air compressors, instrument air dryers, feed pump oil coolers, hydrogen seal oil cooler, and various samplers. The reactor enclosure system removes heat from recirculating pump motors, reactor coolant cleanup system, and the reactor shielding concrete.

Two deep wells drilled on the site supply all well water requirements. In addition to make-up to the two cooling water systems the well water furnishes make-up to the reactor circulating system.

Additional Facilities

The service air and instrument air systems are similar to those in a conventional power plant.

In addition to the normal functions of protecting personnel, equipment, and parts of the structures from abnormal thermal and moisture conditions, the heating and ventilating systems of the Dresden Station have the duty of keeping airborne radioactive contaminants within safe limits and preventing their spread to cleaner parts of the plant. To accomplish this, minimum face velocities of air are provided across openings of cells containing possible contamination sources. Flow is maintained always from areas of little or no activity, and finally to the exhaust stack. Exit velocity at the top of the stack will be at least 3000 feet per minute, which will insure sufficient dispersion to the atmosphere to prevent harmful contamination.

Two oil-fired steam boilers will provide heat for the station. These are located in a bay on the north side of the turbine building at the east end.

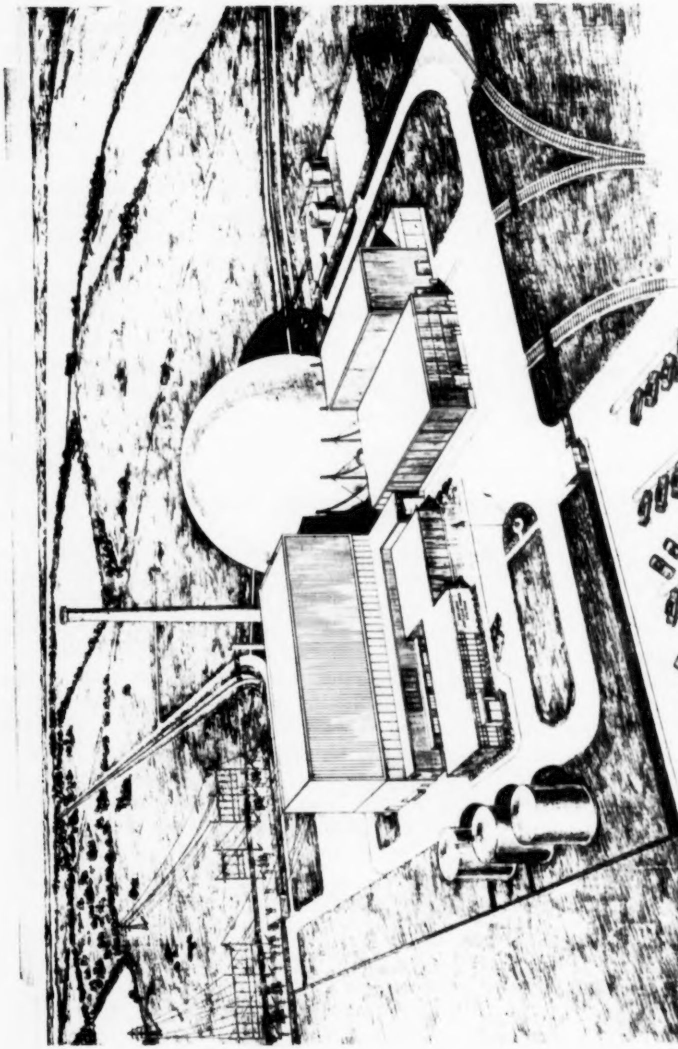


FIGURE I

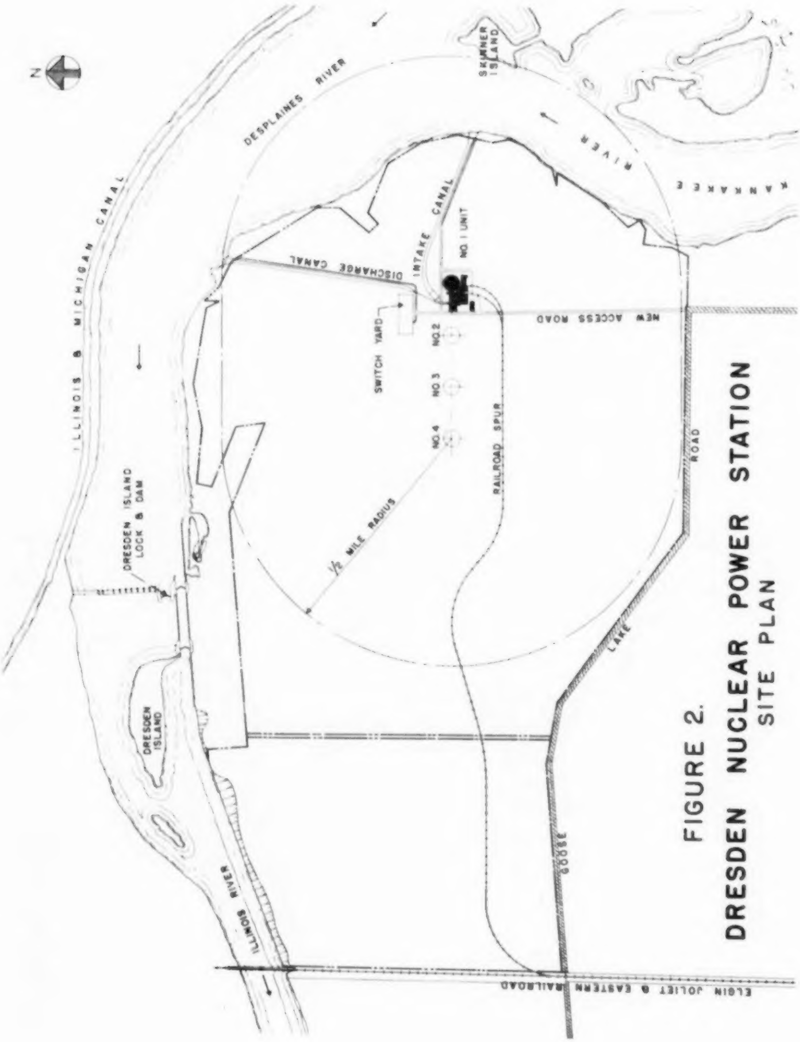


FIGURE 2.
DRESDEN NUCLEAR POWER STATION
SITE PLAN

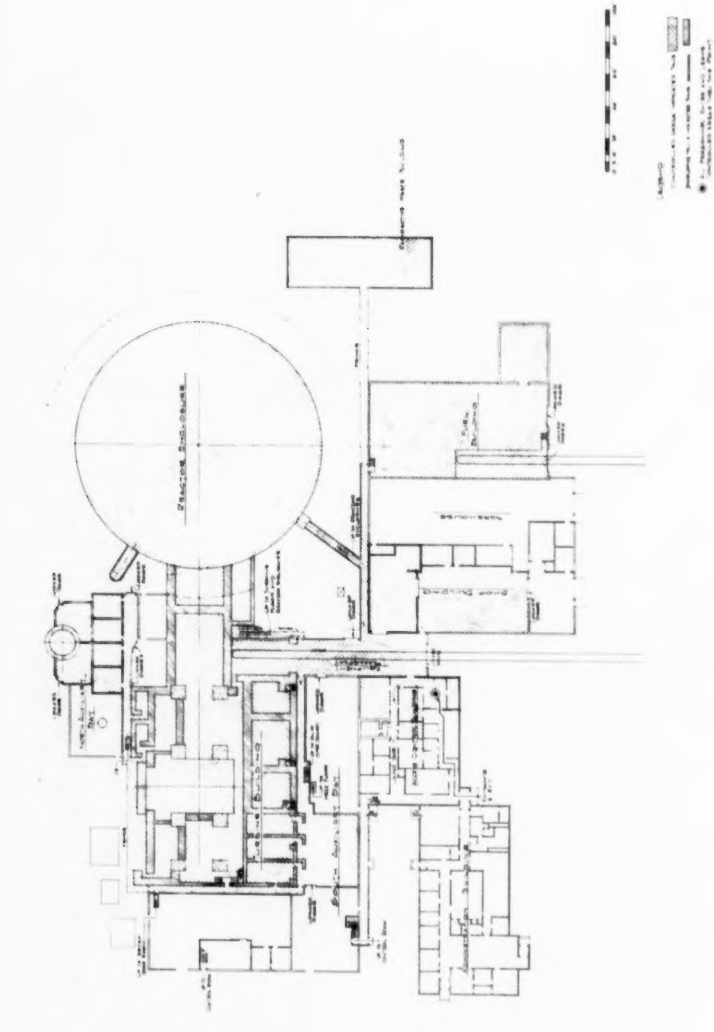
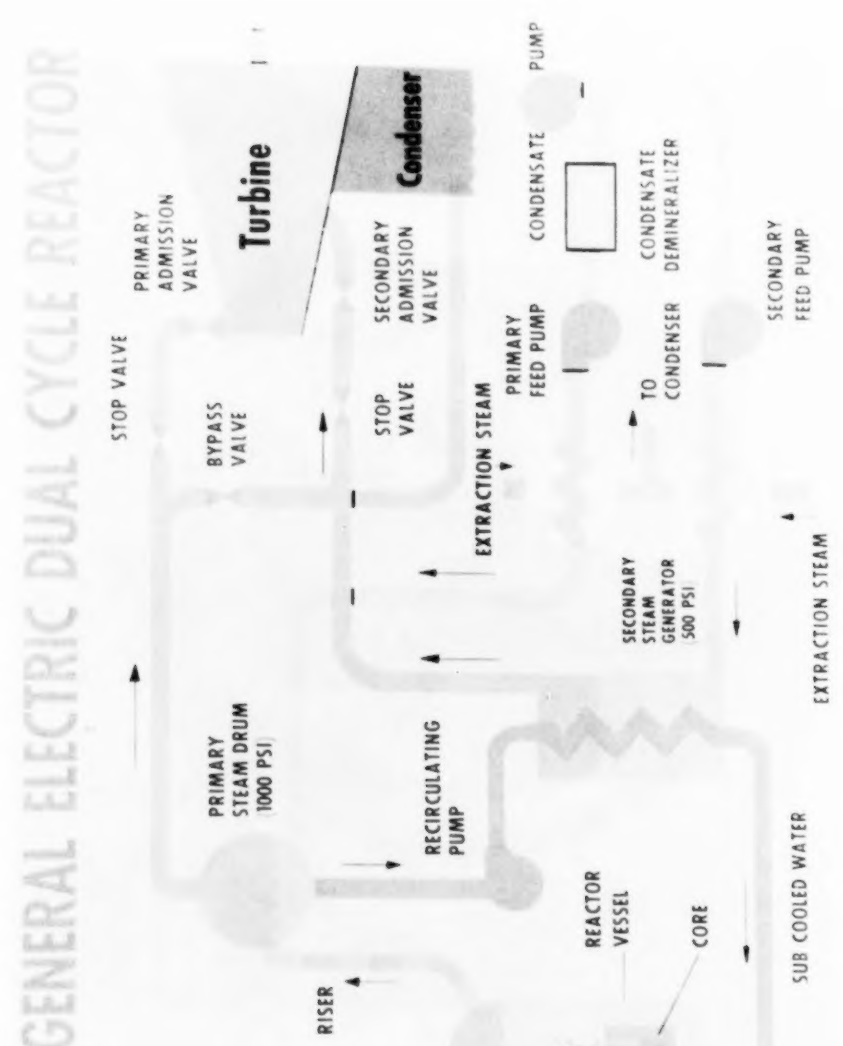
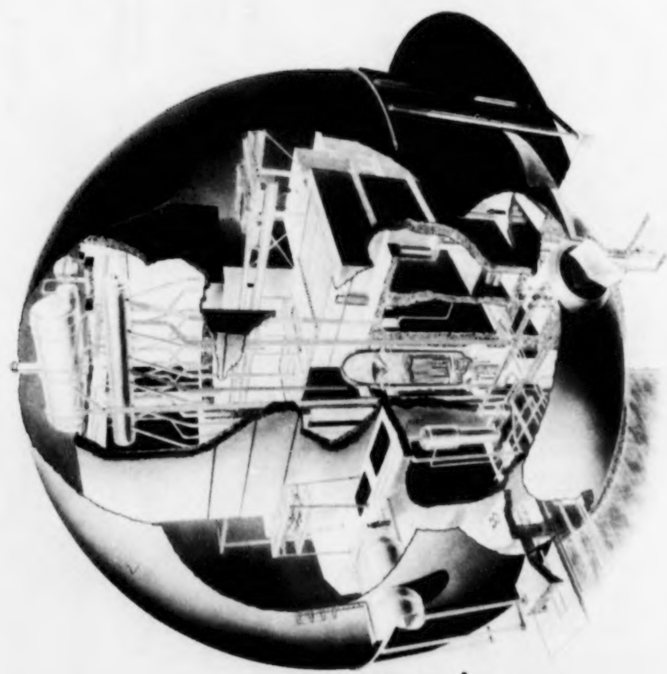


FIGURE 3


GENERAL ELECTRIC DUAL CYCLE REACTOR
STOP VALVE
PRIMARY ADMISSION VALVE
Turbine
Condenser
PUMP
CONDENSATE
CONDENSATE DEMINERALIZER
BYPASS VALVE
SECONDARY ADMISSION VALVE
STOP VALVE
PRIMARY FEED PUMP
TO CONDENSER
SECONDARY FEED PUMP
EXTRACTION STEAM
SECONDARY STEAM GENERATOR (500 PSI)
EXTRACTION STEAM
PRIMARY STEAM DRUM (1000 PSI)
RECIRCULATING PUMP
REACTOR VESSEL
CORE
SUB COOLED WATER
RISER



REACTOR ENCLOSURE CUTAWAY
DRESDEN NUCLEAR POWER STATION

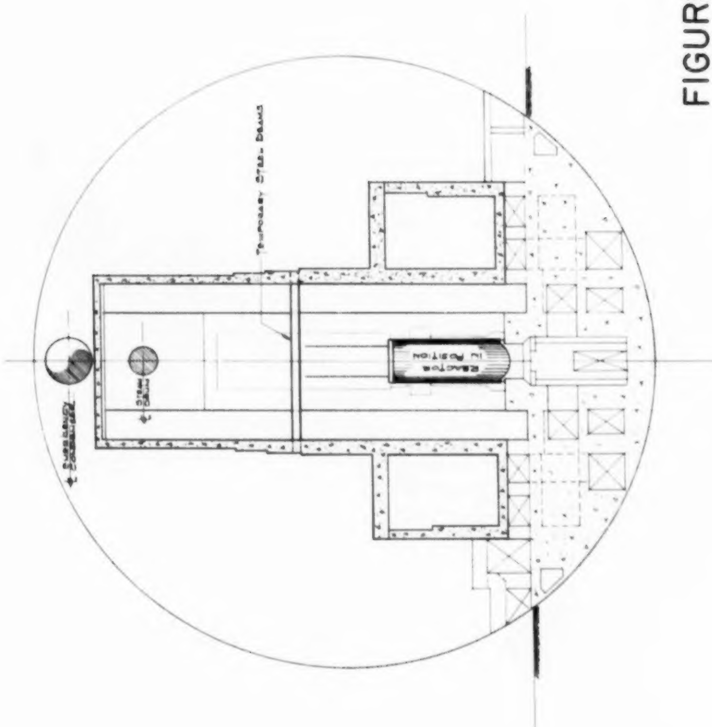


FIGURE 6

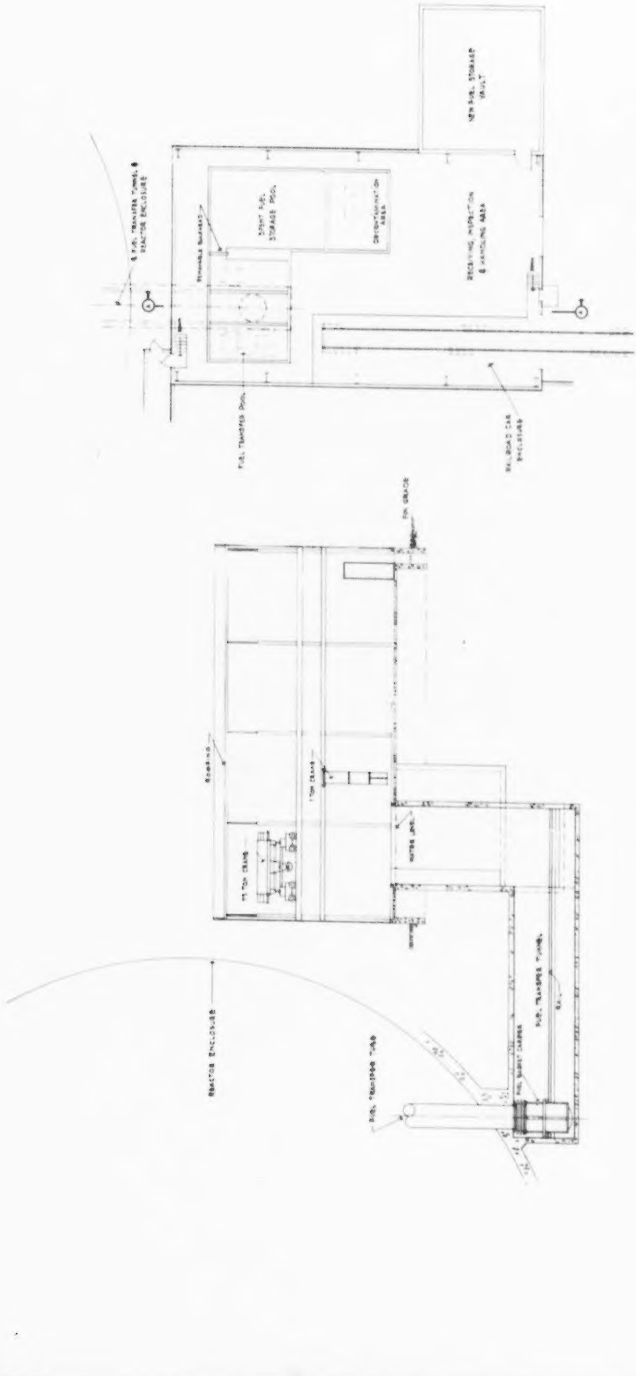

NEW FUEL STORAGE VAULT
SPENT FUEL STORAGE POOL
FUEL TRANSFER POOL
FLOOR PLAN
REACTOR ENCLOSURE
SECTION A-A

FIGURE 7

TURBINE BUILDING CUTAWAY
DRESDEN NUCLEAR POWER STATION

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SPHERICAL CONTAINMENT SHELL OF THE DRESDEN STATION[a]

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(Proc. Paper 1601)

ABSTRACT

The unusual problems encountered in fabricating and erecting a 190' diameter self-supporting leak-tight spherical pressure vessel are presented. Special features which make this 29.5 psig pressure vessel useable as a containment housing around a nuclear power plant include a dual support system, access locks and unusual shell penetrations.

INTRODUCTION

Design considerations for the Dresden Nuclear Power Station determined that a 190' diameter steel spherical pressure vessel most nearly met the many factors that must be considered in providing containment for this particular nuclear reactor and associated auxiliaries. Extensive studies(1) were made by the Bechtel Corporation and the General Electric Company before arriving at this conclusion and before determining the design pressure conditions.

The specifications prepared by these two companies require that the subcontractor be entirely responsible for the design, procurement, fabrication, erection, inspection and testing of the Spherical Pressure Enclosure and its appurtenances so as to meet the specifications and to produce a vessel of the highest integrity suitable in every way for containing all gasses and vapors under the following conditions:

Note: Discussion open until September 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1601 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. PO 2, April, 1958.

a. Presented at Convention of ASCE, Chicago, Ill., February, 1958.

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1. The spherical pressure enclosure is to house a nuclear reactor and miscellaneous appurtenances of a 180,000 kilowatt Nuclear Power Plant.(2) It will normally not be under internal pressure. In the case of certain accidents or malfunction of the power plant, the structure will be subjected to an internal pressure and a coincident internal temperature rise. Due to the great potential hazard to the environs from radioactive material the highest degree of integrity is demanded of the spherical pressure enclosure throughout the entire life of the plant. All portions and details subject to vessel pressure shall have at least as great a margin of safety as the structure as a whole.

2. The spherical enclosure will be based on design conditions as follows:

   a) Design internal pressure is to be 29.5 psig accompanied by a design temperature rise of 250° F.

   b) The large diameter relatively thin spherical enclosure is to be self-supporting and capable of safely resisting all dead and live loads, and in addition should be capable of resisting a minimum external pressure of 1 psig together with the effect of all dead loads.

This paper deals with the designing, fabricating, erecting, and testing of this 190' diameter self-supporting leak-tight sphere. The operating conditions for the spherical enclosure are similar to those of present day power plant buildings except that the sphere must always be ready to act as a pressure vessel. During operation, access for personnel and equipment into the sphere is provided by three locks so arranged that one door is always sealed.

Spherical Shell

The specifications require that this spherical pressure vessel meet the ASME Boiler and Pressure Vessel Code, latest edition, including addenda and Code Cases: Section II, Material Specifications; Section VIII, Unfired Pressure Vessels; and Section IX, Welding Qualifications. The specifications were written before the Code was modified to precisely cover this unusual type of pressure vessel. Since that time pertinent Code Cases have been passed which now make it possible to stamp this vessel. They are:

Case No. 1224-1 defines this type of vessel as follows: "Containment Vessels are those outer vessels which enclose the reactor vessel or portions of the primary coolant circuit or both. The containment vessels are not normally pressurized and are built to contain the lethal radioactive substances that may be released in case of an accident or failure of the reactor vessel or the primary coolant circuit or both."

Case No. 1226-3 recognizes the fact that this type of vessel cannot usually be stress-relieved after completion as required by Section VIII for vessels containing lethal substances. In lieu of stress-relief of the completed vessel this case requires:

1. The plates and forgings must conform to specifications A300 and A350 respectively. These and other materials and the construction must meet certain impact test requirements. Keyhole Charpy

values of at least 15 ft. lbs at a testing temperature of minus 50° F. are required.

2. All doors, nozzles, and opening frames must be preassembled into shell plates and stress-relieved as complete assemblies for butt-welding into the shell.

3. Special consideration must be given to make the design of the reinforcement for large openings (over 40" for this vessel diameter) as strong as the shell.

4. Butt-welds in the shell over 1-1/4" and up to and including 1-1/2", must be pre-heated to 200° F. during welding and welds over 1-1/2" require stress-relief.

5. Shell joints must be double welded butt-joints and must be fully radiographed. All welds such as those around nozzles and opening frames that cannot be radiographed must be examined for cracks by magnetic particle or fluid penetrant method of inspection.

6. Mandatory corrosion allowance in the Code for air, steam or water service does not apply to containment vessels. This vessel can be maintained or protected like other steel buildings.

Case No. 1235 recognizes the hazardous character of the material which might be released and specifies that pressure relief devices are not required.

All of the above items are included in building this sphere as will be seen in the subsequent discussion.

Figure 1 shows the general dimensions and features of the sphere. The sphere is supported entirely on the columns without any internal or external bracing during the first pressure test. For this condition the plate thicknesses in the bottom hemisphere could be about the same thickness as their counter-parts in the upper hemisphere. However, after satisfactory completion of the first tests the lower portion of the sphere will be embedded. Note that the plate thicknesses increase from 1.25 inches at the top to 1.40 inches for the course at the embedment. These thicknesses are determined by combining the internal pressure stress with the stress due to dead and live loads.

Usually the maximum membrane stress is tension in the circumferential direction. If the sphere is cut on a horizontal plane the stress on the cut edge due to dead loads is compression. In a sphere these compressive stresses are balanced by tension stresses in the circumferential direction. After embedment, the internal pressure causes the sphere to rise at the equator, to relieve the load on the columns, and to transfer the entire weight to the spherical shell.

The supports and dead loads are such that the upper 1-1/4" thick plates govern the ability of the sphere to resist external pressure. The Code rules indicate that a 1-1/4" thick sphere with a 95' radius could safely resist a working external pressure of slightly over 2 psig. Deducting the effect of the dead loads (including the weight of the structure) leaves slightly less than 1-1/2 psig allowable external pressure. This exceeds the specification requirement of 1 psig.

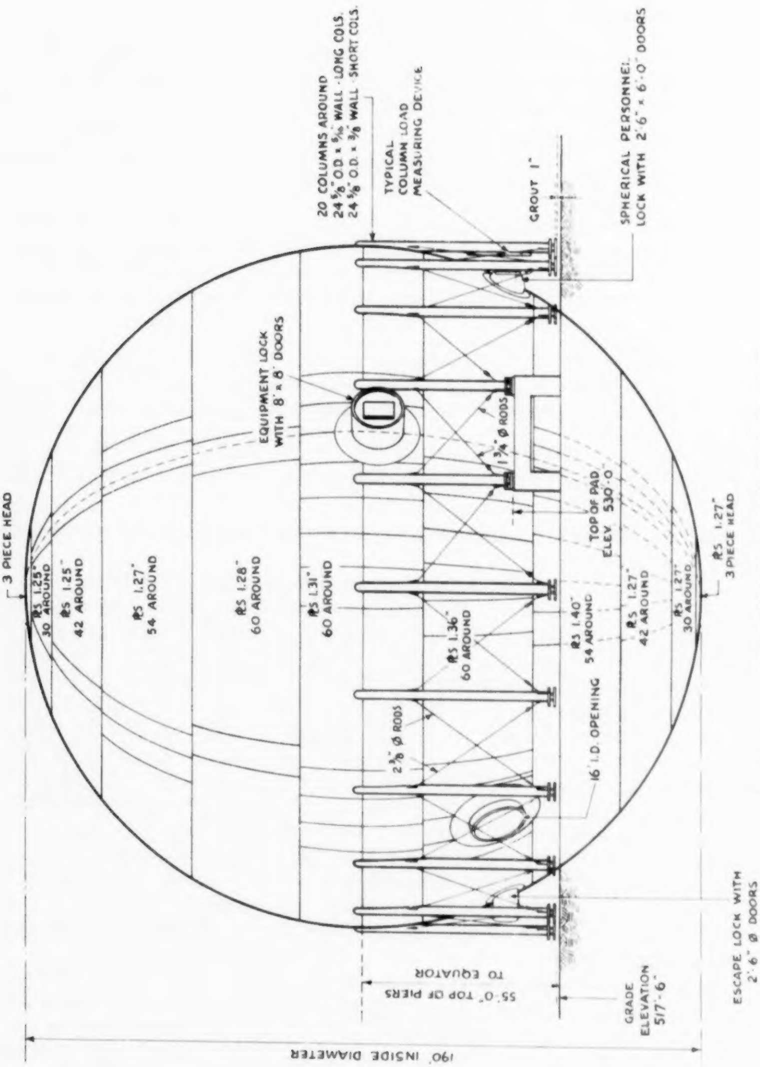

FIGURE 1 ELEVATION VIEW OF 190' DIAMETER SPHERE

During erection, this thin self-supporting shell is very limber and, as will be pointed out later, requires special support. Once the sphere is completed and embedded in its permanent foundation the sphere becomes very stable against wind or earthquake forces. The stresses caused by the dead and live loads and the forces of nature such as wind, snow and earthquake are quite small because of the shell thickness required to resist the pressure stresses. Table 1 gives a summary of the membrane stresses immediately above grade.

TABLE 1. MEMBRANE STRESSES IN 1.40" SHELL JUST ABOVE GRADE

Load	Circumferential Stress	Meridional Stress
Internal Pressure (29.5 psig)	12,000 psi	12,000 psi
Dead Load (Including snow & insulation)	1,250 psi	-1,050 psi
Combined	13,250 psi	10,950 psi
110 mile per hour wind (leeward side)	100 psi	-100 psi
3.3% Earthquake factor (leeward side)	50 psi	-50 psi

When combined, as required by the specifications, the maximum membrane stress at all points is less than the Code allowable of 13,500 psi. This allowable is based upon a 60,000 psi minimum tensile strength steel. A factor of safety of 4 is used to obtain a stress value of 15,000 psi and this is further reduced by multiplying by 90% as required by the Code for 100% radiographed joints.

Fabrication

The steel plates used for the shell and all other pressure parts meet ASTM Specifications A201 Grade B Firebox Quality and are heat treated (normalized) to ASTM A300 Specifications, with Keyhole Charpy Impact Tests taken at minus 50°F. The stress in a sphere under pressure is nearly uniform and the transverse Charpy values will be lower than the longitudinal. Therefore, three impact specimens are taken transverse to the direction of rolling for each plate as rolled. If the average of these tests is less than 15 ft. lbs. the plate will be rejected.

Where plate material can not be used, equivalent material in the pipe, tube or forging specifications will be used. Heat treated plates are available only from eastern mills. Because of this, the fabrication of the heat treated steel (which represents over 95% of the tonnage) is being done in Greenville, Pa. The fabrication of the tower, ladders, etc. (which are A283 Grade C and A7 steel and do not require heat treatment) is being done in Chicago.

Great care is exercised in the fabrication of the plates used in the construction of the sphere. The plates are gaged and carefully inspected for laminations and surface defects. Wherever possible, defects are repaired using careful procedures approved by the inspector or if this is not possible or practical the plates are rejected. The plates are formed using a hydraulic press. All plates are treated using the Horton Pickling Process which consists of immersing the plates in baths of sulphuric acid, wash water, and phosphoric acid. The plates, except for the surfaces immediately adjacent to the edges prepared for welding, are given a shop coat of paint immediately after pickling. Considerable time is spent removing mill scale from the plates. This seems to be a special problem with thick normalized plates.

Nearly all of the shell penetrations and all of the column stubs are shop welded to shell plate sections. Most of the welding is done in the field; however, the same careful welding procedures that are discussed later under Field Erection are also used in the shop. Each of these shell plates involving weldments is thermally stress-relieved in a shop furnace. Figure 2 shows several of these plate sections on the stress-relieving car.

The three locks which will be discussed later are completely assembled in the shop. Two of these locks are welded up into complete units including the adjacent shell and its reinforcement. The completed units are shipped in one piece to the field. The third lock being larger is welded up into four separate sections. These will be shipped individually to the field, for final assembly.

Supports

The sphere is supported during erection by twenty 24'' diameter tubular columns braced by diagonal rods. The columns are supported by individual concrete piers resting on sandstone. The lower column sections are erected first, and are followed by erection of the equator plates which have column stubs attached to every third plate. Figure 3 shows the equator course at the time the close-out section is being erected.

Two columns are shorter than the others because of a concrete shielding tunnel surrounding the main steam lines. These two columns are supported by the tunnel, and are slightly heavier than the other eighteen because they carry the load of the equipment lock located midway between them. These columns are of 3/8'' nominal thickness and are designed to carry 417 kips dead load plus 62 kips from an assumed 110 mph wind load. During the overload air test, an additional load of 29 kips is imposed on each column by the 580,000 pounds of air which are pumped into the vessel.

The eighteen long columns are of 5/16'' nominal thickness and are designed to carry 348 kips dead load plus 62 kips wind load. Again, they support an additional 29 kips during the air test.

Having two short columns requires special consideration in designing the four short diagonal tower rods attaching thereto. Assuming uniform horizontal deflection of the column tops under the imposed wind loading, these short rods must be strained appreciably more than the long rods. The long rods are made of 2-3/8'' diameter A7 material. The short rods are made of 1-3/4'' diameter high strength steel so the maximum rod pull is the same as that of the long rods, permitting near duplication of the end connections.

The adjustable column base, the jacks, and the load weighing device are shown in Figure 4. Should a column become overloaded, it can be jacked

Figure 2

Shell plate weldments enroute to shop stress relieving furnace

Figure 3

Equator course on 20 tubular columns

down. Conversely, if a column is not carrying its share of load, it can be jacked up. Column loads are evaluated by indicators on the hydraulic jacks and by column load-measuring devices.

For reasons to be discussed in subsequent paragraphs, it becomes necessary to have dual supports for the sphere, namely the embedment and the columns. The column bases must be grouted in the early stages of erection to support the steel weight. The adjustable column feature permits proper distribution of the total structure load to each of the two support systems. The adjustment feature also provides for correction in the unlikely event of differential settlement of the column piers.

The column load-measuring device consists of a 2 inch diameter pipe approximately 19 feet long fixed to the column at the top end and free but guided at the bottom. A stainless steel stud is attached to the bottom end of the pipe and a similar stud is attached directly below it to a bracket on the column. A gage gap of approximately 6-3/8 inches separates the two. The change in the gage length is determined by measuring the gage gap to the nearest one-thousandth inch with a dial type extensometer. As the column is loaded the gap decreases. The change in column length over the 19' gage length is converted into stress and then into the direct compression load.

The sphere without embedment cannot support the heavy internal loads from the power plant equipment. Accordingly, the lower portion (39 feet) of the sphere is embedded in concrete. The internal loads are transferred by bearing through the inside concrete, the shell, and the outside concrete into the rock substructure. An air test is conducted before the sphere is embedded in order to permit full inspection of all welded seams during the test.

Figure 5 illustrates the general detail of the transition embedment. A sand pad 8 feet deep and of variable radial thickness provides transition easement at grade where the sphere emerges from the embedment. The sand is backed up by a concrete ring bearing on rock. The compressibility of the sand permits the sphere to expand gradually. A mastic easement or a tapered transition will be provided between the embedment concrete and the shell permitting the vessel to grow smoothly into the sand pad.

A drain trough is provided at the bottom of the sand. When the installation is completed, the run-off water will be flashed out of this area. The insulation on the inside of the shell functions to establish an acceptable temperature gradient. The gradient is located so that the maximum bending stress from pressure does not combine with the maximum bending stress caused by the temperature gradient (due to 250°F. rise). The principal reason for the external insulation is for environmental temperature control. A weather skirt surrounding the vertical 12' of sphere immediately above grade completes the outside insulation.

The theoretical radial growth of the sphere due to 29.5 psig internal pressure is 0.33" and a 250°F. temperature rise increases this to 2.24". Consideration of this growth confirms the need for special embedment considerations at grade. The restraint at grade is similar to the restraint of the bottom of an oil storage tank or of a circumferential stiffener around a penstock. The combined membrane stresses at a point just above grade are summarized in Table 1 (Page 5).

At the time the sphere is pressurized to 29.5 psig and has undergone a temperature rise of 250°F., the maximum meridional combined stress (membrane plus bending) will occur at the top of the inside insulation (21,250 psi on the outside surface) rather than at grade. This is the point of maximum

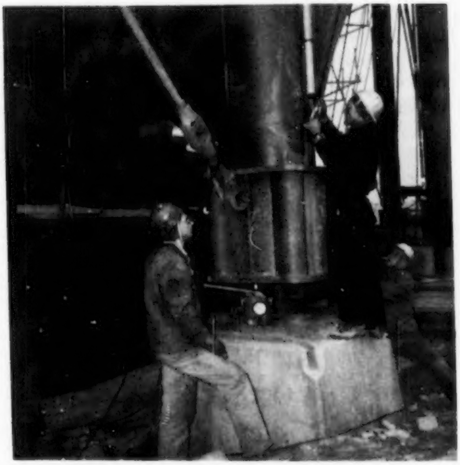

Figure 4

Adjustable column base and load measuring device

Figure 6

50-ton derrick and welding table

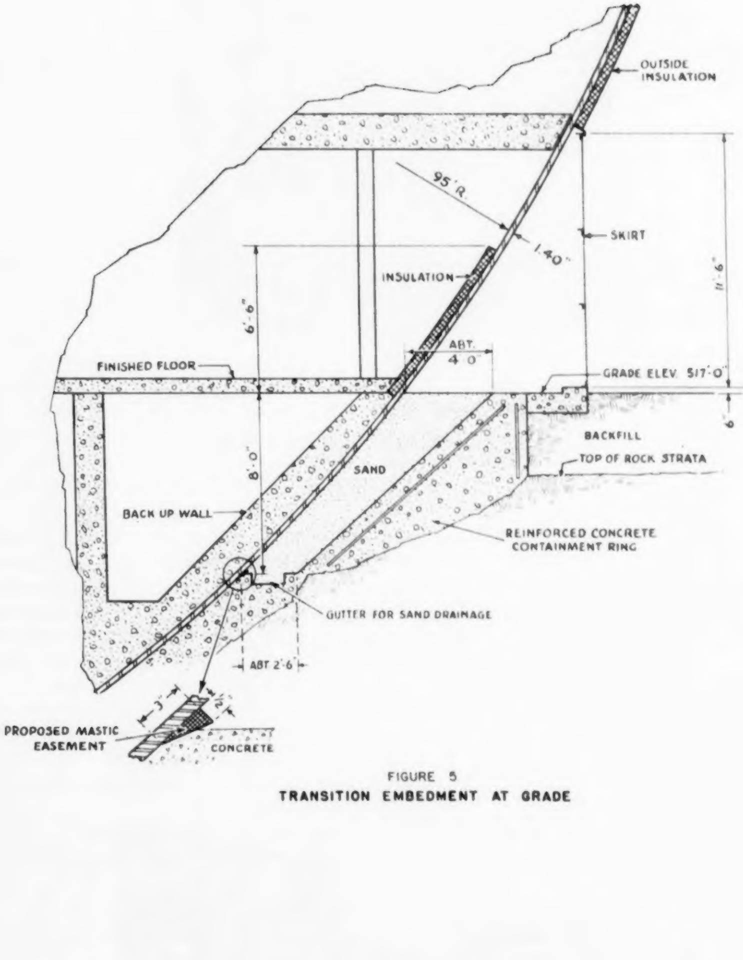

FIGURE 5
TRANSITION EMBEDMENT AT GRADE

bending stress ($\pm$9,655 psi) due to the 250°F. temperature rise. The bending stress due to internal pressure is a maximum ($\pm$2,260 psi) at the bottom of the sand pad where the bending stress due to temperature is negligible.

Consideration was given to removing the columns after embedment; however, the 1 psig external pressure requirement superimposed on the dead load of the vessel would produce excessive compressive stresses in the thin shell at grade. The plate thickness required to develop these loads at allowable design stress exceeds 1-1/2'' and the welded seams would require field stress-relief to meet Code requirements. This would be impractical and therefore the columns will become part of the dual permanent support system. The diagonal tower rods are to be removed after embedment since the structure can then readily develop the lateral shear loads.

Field Erection

While a certain amount of standard equipment can be used to advantage in erecting a special structure such as this, a large percentage of it must be custom built.

All of the steel is erected by a 100-ton guyed derrick, located on the vertical centerline of the sphere. The derrick consists of a 170-foot mast and a 135-foot boom mounted on a tower 210 feet high. The base of the tower is 40-feet below grade. Hoisting power is supplied by a multi-drum hoist. The four post derrick tower converges into a double trunnion at the base to provide end rotation in any direction. This prevents the development of any end moment that might otherwise exist at the time the eight 2-1/4'' diameter cable guys are developing the horizontal load of the boom.

Material is transported into an assembly yard by rail. It is unloaded and sub-assembled using a 50-ton derrick. Figure 6 shows the assembly yard derrick lifting a four plate section from a welding table. A portion of the 100-ton derrick boom can be seen in the upper right hand corner. The boom radii of these derricks overlap sufficiently to pass lifts directly from one to the other.

Several individual shell plates as large as 10' x 35', and weighing as much as ten tons each, are automatically welded together on the ground and hoisted into plate in a single lift. The automatic welding is performed on double acting, adjustable tables and all seams are preheated in accordance with Code requirements. Figure 7 shows a typical four plate assembly mounted on one of the adjustable welding tables. The automatic welding machine and operators can be seen completing the back-side welding on the section. When the back-side welding is complete the section is placed on a stationary table where the welding is 100% radiographed and repaired where necessary. The automatic welding is by the submerged arc process.

After the multiple-plate sections are fitted together one to another in a particular course, the joints between sections (radial seams) are double-butt welded by hand using the shielded metal arc process. After welding from one side, the back-side is carbon arc gouged and inspected by the magnetic particle method. As with the automatic welding, all seams are preheated to 250°F. before welding is done, and the completed weld is 100% radiographed.

The sequence of erection is from the equator to the bottom and then from the equator to the top. The horizontal welds between courses (girth seams) are also hand welded. All hand welds are made using low hydrogen electrodes.

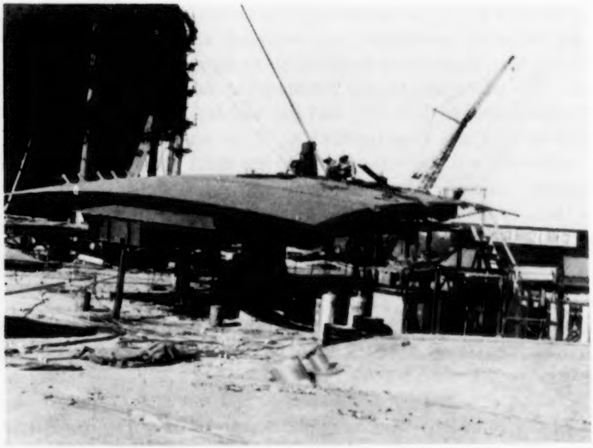

Figure 7

Automatic welding on adjustable table

Figure 8

Temporary supports for upper plate sections

To assure the vessel is built to a truly spherical contour, the equator course is held round by a spider ring supported off of the derrick tower. The subsequent courses erected below the equator are suspended back to the tower by hangers of predetermined length. The courses above the equator are supported back to the tower by hogged booms of predetermined length. Figure 8 shows the arrangement of these booms with a portion of the upper hemisphere erected.

This dimensional control and periodic tape checks insure that the vessel will be built to a constant spherical radius. The Code states that the inner surface of this vessel shall not deviate from the specified shape by more than 1.25 percent of the inside diameter. This gives a permissible out of roundness of 28.5 inches, which is several times greater than that anticipated. To control local shell contour, the specifications require checking the curvature with a 15 foot template. When entirely on a single plate and no closer than 12 inches to a welded seam, the permissible deviation from the template is 0.50 inches. When the template is placed across one or more welded seams the deviation shall not exceed 1.00 inches, after allowance for any weld reinforcement.

During erection, the partially completed upper hemisphere has a tendency to lay in from its own dead load and is further pushed in by wind loading. Special erection devices are used to stabilize the upper end of these courses during erection.

There are 492 individual shell plates in the sphere and approximately 17,200 feet (3-1/4 miles) of welded seam. The geometric volume is 3,600,000 cubic feet and the surface area is 113,400 square feet (2-1/2 acres). The completed vessel weighs approximately 3,500 tons.

The required stress-relief of penetration weldments is performed in the shop on all but the sixteen foot diameter equipment opening. The 26'-7" overall diameter of this weldment precludes built-up shipment and thus it requires a special field stress-relieving furnace. The complete assembly including the cover was heated to 1150°F. minimum for a holding time of three hours and then cooled down at a prescribed rate. Careful control of distortion on finished gasket surfaces is required to insure proper matching. The details of this opening will be described in the next section.

Penetrations

There are a total of 132 permanent shell penetrations ranging in size from 1/2 inch to 16'-0" in diameter. Various connecting lines between the sphere internals and the power plant buildings pass through these penetrations. Expansion joints will be installed in certain connecting lines to provide flexibility and to reduce loads on the sphere resulting from piping movements.

All corner welds joining necks to the shell and reinforcing plates are full fusion welds. The detail of a typical penetration weld is shown in Figure 9. At least one end of each penetration is prepared for welding to the connecting line or expansion joint. The connecting welds are made after the sphere has passed the first test. A typical expansion joint connection is shown in phantom. Temporary test covers are used to blank off these penetrations during the first air test.

Three of the major penetrations are air locks, and are described in the next section of this paper. The largest and most difficult opening to provide

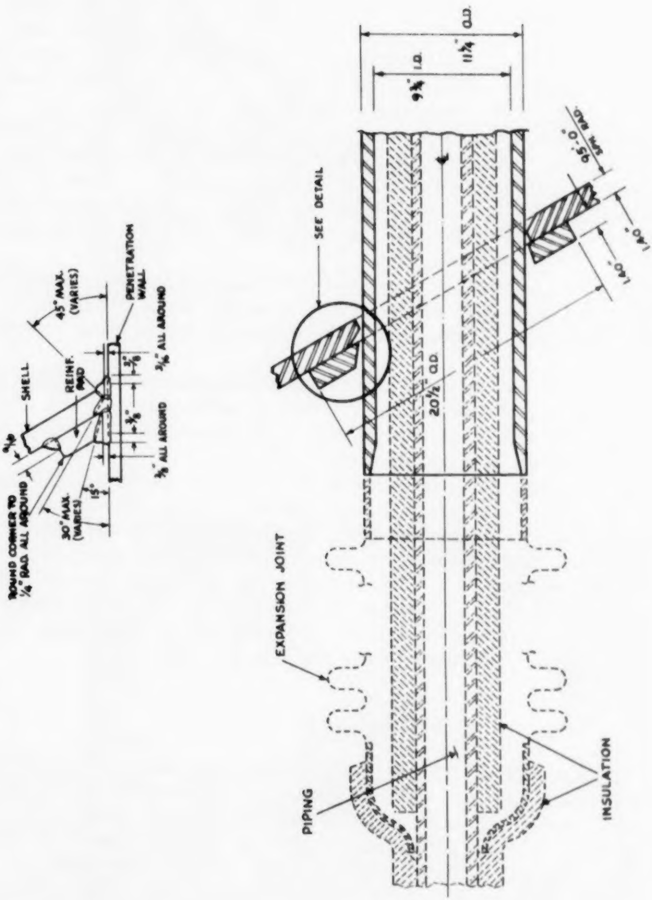

FIGURE 9
TYPICAL PENETRATION DETAIL

is the 16'-0" diameter permanent bolted opening. It is centered about 17'-6" above grade so that it does not complicate the stress pattern at grade. All of the penetrations are purposely located outside the area just above and below grade.

Considerable thought was given as to how the 16 foot diameter opening could best be provided. The problem was resolved into four basic phases, namely:

a) Design of the shell reinforcement and cover to satisfy Code material and stress-relief requirements;

b) Procurement of special materials;

c) Development of the gasket, seal weld, and bolting details;

d) Development of an inside cover handling device.

Figure 10 illustrates the general arrangement of the reinforcement, the bolted cover, and the handling device.

The reinforcement consists of a 16'-0" I.D. x 17'-0" O.D. A350-LF-1 seamless forged ring 1'-6" long, welded to a 3 inch thick by 26'-7" O.D. dished insert plate. Procurement of the forging as an integral ring proved to be a major problem. A leading manufacturer of forged rings accepted the order after a mutual understanding was reached regarding practical dimensional tolerances. The finished weight of the ring is about 10 tons. The 3 inch insert plate appears to be the maximum thickness which a mill will roll A201 plates to the A300 specifications requiring guaranteed 15 foot-pound transverse Charpy tests at -50°F.

The field welding of the insert plate to the forging was done on an around-the-clock basis. Inasmuch as this weld could not be radiographed, the welding procedure was particularly detailed and included considerable magnafluxing of interpass beads. The welding was also programmed to minimize distortion of the finished gasket surface on the forging. The splices in the 3" insert plate were 100% radiographed. The bolt lugs and seal weld angle were also welded to the forging at the site. The bolted cover was rigidly locked to the reinforcement during stress-relief so that it would assume the same plane as the ring should the ring distort. The completed reinforcement weldment and cover were satisfactorily field stress-relieved and the gasket faces remained remarkably plane.

A 1-1/4 inch square gasket mounted in the cover is drawn tight by 1-1/2" diameter eyebolts on 12 inch centers. The pressure on the cover inside the sphere further loads the gasket when the vessel is pressurized. Should difficulty be experienced in obtaining a satisfactory degree of tightness with the gasket alone, a seal weld detail will be utilized to effect a tight seal. The seal weld detail also provides a means of testing the tightness of the gasket without pressurizing the entire sphere.

The bolted cover is removed by pulling it away from the shell with jacks mounted on an inverted A-frame. The frame is suspended from an overhead trolley beam along which it is also translated (after jacking) away from the opening. The power is provided by a chain hoist. The procedure is reversed to replace the cover.

There are two other relatively large openings each about 40" in diameter. One is the fuel handling penetration which lines up with an underground tunnel and is located below grade. The other is an emergency condenser vent at the

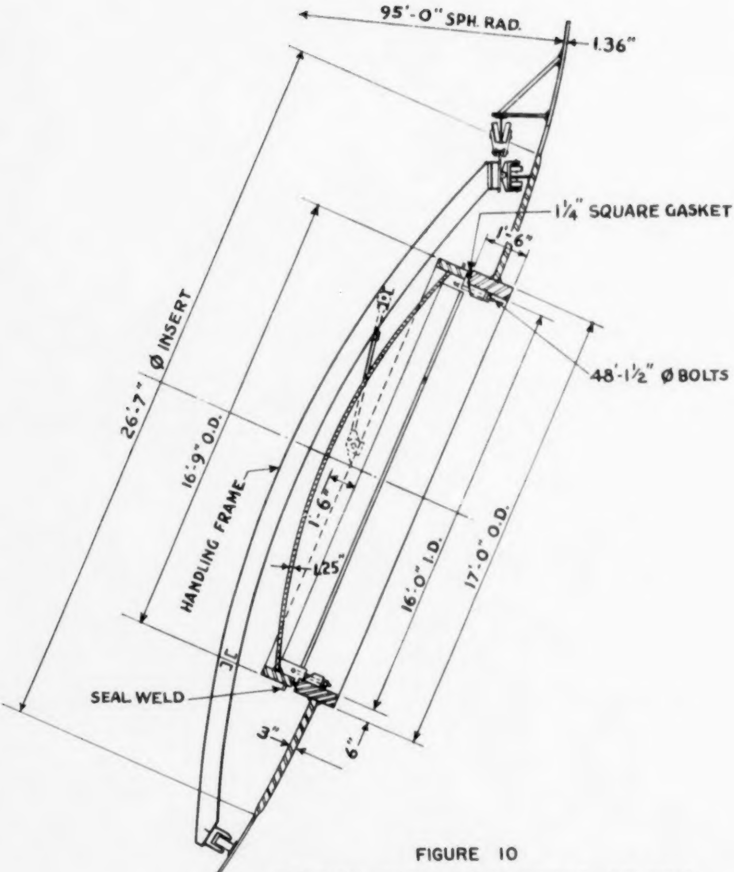

FIGURE 10

SECTION THRU 16' DIAMETER OPENING

very top of the sphere. The fuel handling penetration and other penetrations over 40 inches in diameter are reinforced in accordance with Code rules, except that the required reinforcement is increased by 40 percent to minimize secondary stresses around these shell openings.

All penetrations are made from one or more of the following materials: A201 Grade B Firebox A300 plate; A333 Grade C pipe; or A350-LF-1 forgings. In each case the required Charpy impact tests were made. A minimum spacing of twice the average opening diameter was maintained between all permanent penetrations.

After the sphere is erected and tested, two large temporary openings will be cut to facilitate internal construction and entry of the reactor components. The smaller of these openings is a true circle, 24' in diameter near grade and is reinforced with a heavy angle. The other large temporary opening is 22 feet wide and 29 feet high (vertically) with rounded corners. It is also at grade and is reinforced completely along all edges. The latter opening will be cut after embedment whereas the 24 foot diameter opening will be cut immediately after the first test. After the internal construction is complete and the internal equipment placed, these two cut-outs will be replaced and their reinforcements removed.

While not in the category of penetrations, scaffold attachment clips are provided on approximately 8 foot centers on the inside of the upper hemisphere of the vessel. Also, an outside stairway from the equipment lock to the top of the sphere is provided. Various studs are provided on the exterior of the sphere for attaching insulation, flashing, drain troughs, etc. All attachment welds to the sphere are made with low hydrogen electrodes.

Air Locks

Three pressure locks are provided for personnel and equipment access through the containment vessel shell. The locks are chambers with pressure tight doors connecting the interior of the containment vessel with either adjacent buildings or with the "out-of-doors."

The escape lock shown in Figure 11, is 4'-9" in diameter by 10'-0" long, with 2'-6" diameter doors in each end bulkhead. Its function is to provide an emergency exit for personnel in the vessel. This lock is manually operated only.

The personnel lock illustrated in Figure 12, is of spherical construction, 12 feet in diameter with 2'-6" by 6'-0" doors in each end bulkhead. It provides the primary passage way for personnel entering or leaving the containment vessel. This lock will normally be operated by electrical push button controls. However, it may also be operated manually at any time or in the event of electrical power failure.

The equipment lock, Figure 13, varies in diameter from 11'-0" to 14'-0", and is 31'-9" long. It has 8 foot x 8 foot doors in the end bulkheads, and will be used for passage of maintenance personnel, trucks, and other equipment into or out of the vessel. It will normally be push button operated, but is also provided with manual controls which may be used at any time in lieu of the electrical system.

With either the manual or electrical operation, each door can be operated from either side of the bulkhead in which it is placed. Doors in each lock are mechanically interlocked so that both doors cannot be open at the same time.

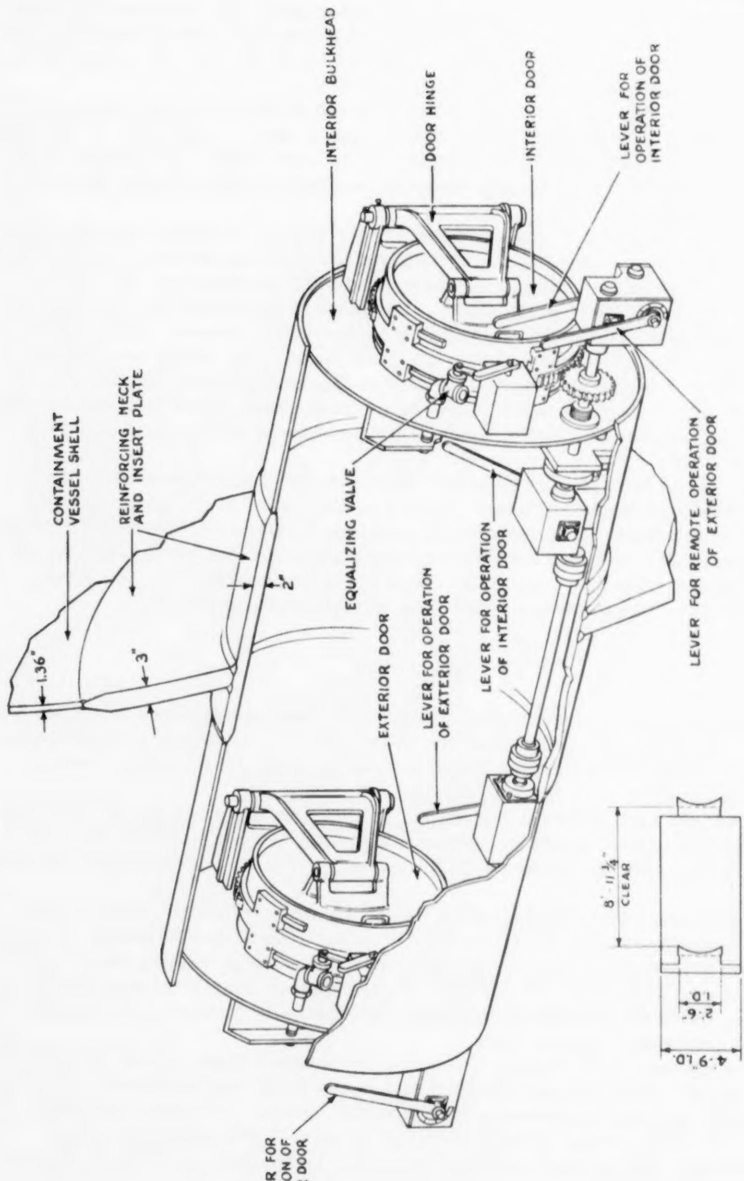

FIGURE 11
ESCAPE LOCK WITH 2'-6" DIAMETER DOORS

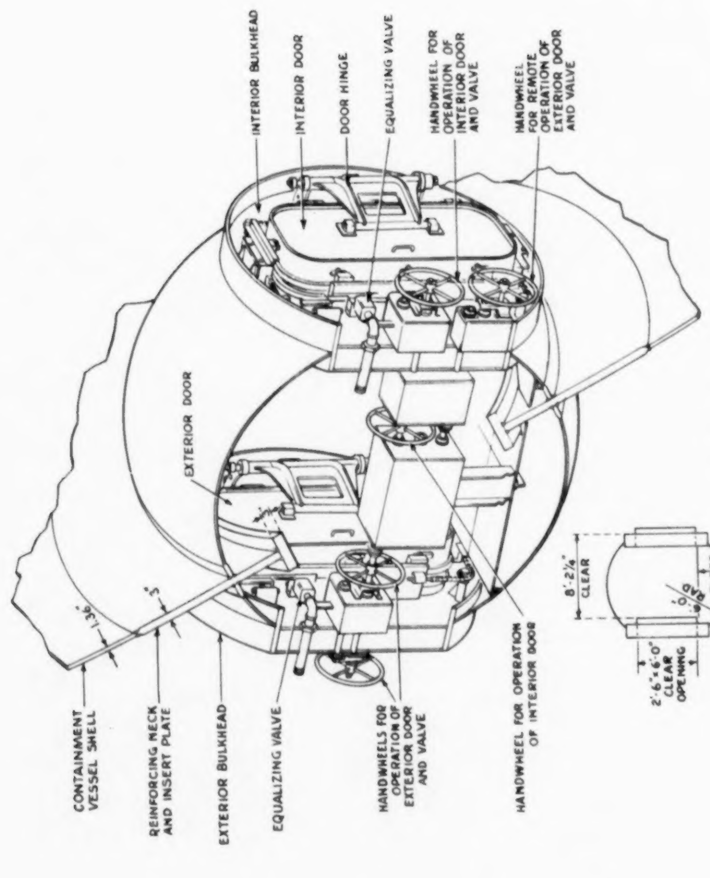

FIGURE 12

SPHERICAL PERSONNEL LOCK WITH 2'-6"×6'-0" DOORS

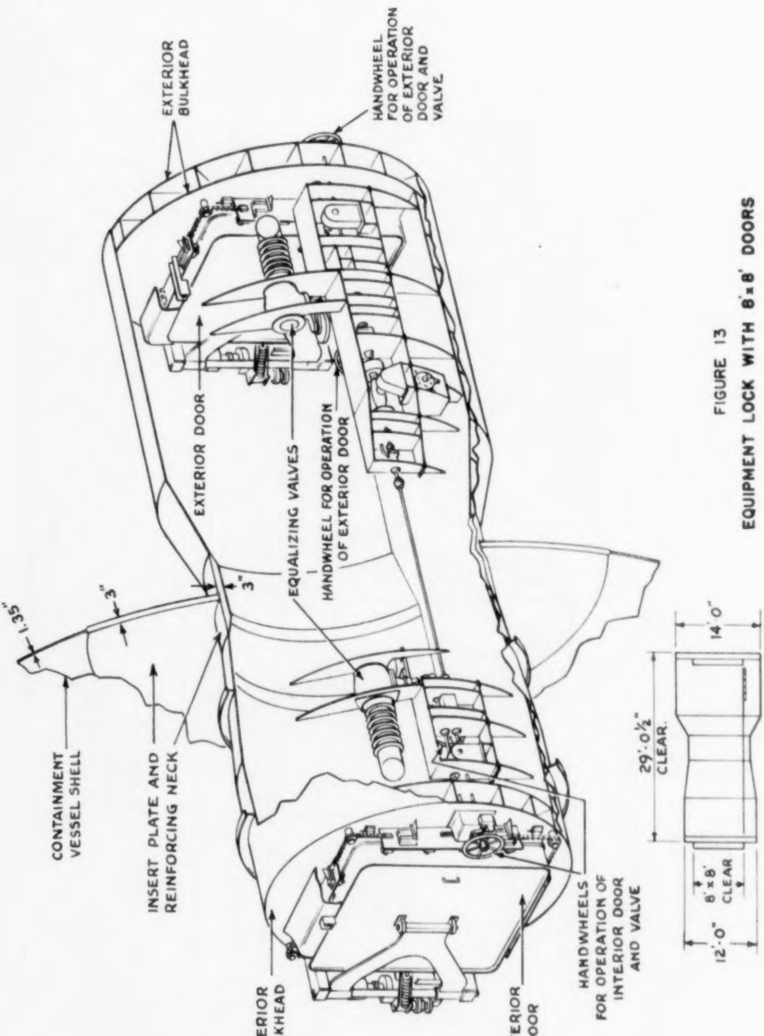

FIGURE 13

EQUIPMENT LOCK WITH 8'x8' DOORS

In the personnel and equipment locks, the doors are also electrically interlocked. One door cannot be opened unless the opposite door is sealed. Each door opens toward the inside of the containment vessel.

A pressure difference between the lock and the vessel or the lock and the atmosphere, will produce forces against the doors. Two pressure equalizing valves are installed in each lock to provide for equalizing the pressure across either door. The valves are also mechanically interlocked and synchronized with door operation so that both valves cannot be open at the same time and so that one door cannot be opened unless the opposite valve is closed.

The plug type valves are sized to bleed a pressure of 20 psig to atmospheric in a period of 15 seconds. The escape lock is equipped with 2" diameter valves; the personnel lock has 2-1/2" diameter valves; and 10" diameter valves are provided in the equipment lock.

It is anticipated that the interior door of the escape lock will normally be left in the open position. This will facilitate prompt egress from the vessel in an emergency.

The procedure for passing through the personnel lock using the hand wheel operators, can be described by referring to Figure 12. A person wishing to exit from the vessel will approach the interior of the lock. He will begin to operate the interior door hand wheel which is clearly marked by an indicator in contrasting colors.

Operation of the hand wheel will first engage the mechanical interlock. This mechanism locks the hand wheels which control the opposite or exterior door. Thus, from this time until the interior door and valve are again sealed, the exterior door and valve are locked in the pressure tight position.

Second, as the hand wheel is rotated, the interior equalizing valve is opened. This equalizes the pressure in the lock with that in the vessel.

Further rotation of the hand wheel will perform the third function which is the unseating of the door from its gasket. The door is mechanically pushed out a sufficient distance to provide a space between the door tongue and the rubber gasket. This space furnishes an additional means of pressure equalization across the door.

The fourth and final operation accomplished by continuous rotation of the hand wheel is that the door is swung open on its hinges. Here the person has an option. Since the door is "free wheeling" he can either pull the door open by the handle on the door, or swing the door remotely with the hand wheel.

The person may now pass into the lock. Before he can gain access to the adjacent building he must close the interior door behind him, pass through the lock, and open the exterior door.

Operation of the interior door hand wheel inside the lock, will reverse the four steps described above. The interior door and equalizing valve will be in the fully closed and sealed position before the interlock is returned to the position where the exterior door hand wheels are unlocked.

With the interior door closed behind him, the person proceeds through the lock to the exterior hand wheel. Operating this hand wheel performs the same operations in identical sequence as described above for the interior door. The exterior door is now in the open position and the person may pass from the lock to the building.

Personnel will be instructed to always close the exterior door behind them as they leave the lock. If the exterior door were inadvertently left in the

open position, the interlock would be engaged. Thus, the interior door hand wheels could not be operated. However, it will be noted that two hand wheels are provided on the vessel side of the interior bulkhead. The extra hand wheel is provided for emergency and will perform the four operations on the exterior door. If the exterior door is open at a time when quick exit from the containment vessel is necessary, it can be swung shut on its hinges, the door and valve sealed, and the interlock returned to a position which will allow operation of the interior door. With this feature, personnel in the vessel are protected against the possibility of being locked in.

A person wishing to enter the containment vessel through the lock, will employ the same principles of hand wheel operation as described for leaving the vessel.

The escape lock and the equipment lock have the same features of interlocking, equalizing valve and door seating, and door swinging as the personnel lock, with one exception. The exception is that the hand wheel operation on the equipment lock does not include swinging the door on its hinges. An electric motor drive is provided for this. The door is "free-wheeling" and can also be swung by hand. Also on the equipment lock the additional hand wheel on the vessel side of the interior bulkhead for remote operation of the exterior door is not provided.

Pointer indicators at each hand wheel on the two large locks, indicate the various positions of doors and equalizing valves. In the escape lock, each hand lever is swung through a total arc of 120° to perform all four operating functions on one door. A person can perform all functions necessary to pass through this lock in a matter of seconds. Pointer indicators are also provided on each of the levers to show door and valve orientations relative to lever travel.

The electrical systems on the personnel and equipment locks are designed to perform (by pushbutton) all of the operations which may be accomplished by the hand wheels. There are push button stations to correspond with each of the hand wheel locations. A single push button operation performs all of the four functions required for complete door and valve operation.

Indicator lights are located at each of the push button stations. These lights are color coded and for each door, they indicate the following conditions: the valve closed and the door sealed; the valve open and the door sealed; the door swung shut but not sealed; the door in the open position; and "door in use" pilot lights are provided at exterior and interior stations.

Push buttons are furnished so that a person on the outside of the lock who wishes to enter the vessel can operate the interior door in the event that it happens to have been left open. Also a person inside the vessel with intention to go through the lock, may, by push button close the exterior door in the event that it happened to be in the open position.

The structural design requirements of the locks can best be described by visualizing a lock with a particular orientation of its doors.

First assume that the exterior door of the lock is open to the atmosphere and the interior door is in the closed or pressure tight position. In this condition, the design pressure of 29.5 psig in the containment vessel will impose an external pressure of equal magnitude on the portion of the lock which lies inside the vessel. Also, in this condition, if the external pressure of the containment vessel is 1 psig, the portion of the lock inside the vessel will be subjected to a net internal pressure of 1 psig.

Next assume that the interior door of the lock is open and the exterior door is in the closed or pressure tight position. In this case, that portion of the lock which lies outside of the containment vessel will be subjected to the same pressure as may exist in the containment vessel itself.

The various design pressures described in these conditions produce membrane and bending stresses in the lock shells, doors and end bulkheads. Certain stresses resulting from dead and live loads in the lock are superimposed on stresses due to pressure only.

Each lock is supported entirely by the containment vessel shell. The portions of the lock which lie inside and outside of the vessel are cantilevered from the portion actually built into the shell.

To insert the lock assembly in the containment vessel shell, it is necessary to cut large holes in the shell plates. These cutouts are reinforced to maintain continuity of membrane action in the shell. The reinforcing is also designed to distribute the dead and live load of the lock over a large area of vessel shell.

The door hinges are located on the bulkheads such that both interior and exterior doors swing open toward the inside of the vessel. The advantage of this arrangement is that the design pressure of 29.5 psig will furnish a gasket seating force on the interior door or on the exterior door if this pressure is allowed to build up in the lock. The sealing mechanism is not required to hold the door in a pressure tight position against the high pressure load. This provides an additional safety feature and lessens the mechanical advantage requirements of the mechanism.

The door locking mechanism is designed to hold the door in a pressure tight position against an external or internal pressure of 2 psig in the lock. With both doors in the sealed position, and when pressure in the vessel is atmospheric, the lock may be pressurized to 2 psig internal or external.

The principal requirement for the locking mechanism and the seal is that it must be possible to obtain a pressure tight seal with a minimum of manual handle effort and in as short a time as possible. Operation of levers or hand wheels must be simple and quick in order to minimize possibility of confusion or delay in emergency conditions. On all of the locks a manual handle effort of 20 lbs. was used in designing the mechanical components of the locking mechanism.

These criteria led to the selection of a tongue and groove type seal in which a soft rubber gasket (40 durometer) is used. A pressure tight seal is obtained with a minimum gasket seating force. Tests were conducted in which the actual seal detail was subjected to design pressure and temperature conditions. In these tests the gasket and surrounding metal were raised to 350°F., held at 350°F. for four hours and cooled to ambient temperature in the ensuing 24 hours. The gasket seal was required to contain a pressure of 37 psig while at 350°F. and various pressures below 37 psig as the temperature was lowered. Subsequent to the test, the gasket was again pressurized to 37 psig. Seven days later the 37 psig pressure application was repeated and the seal found to be pressure tight.

In each of the locks, the locking mechanism is designed to force the door tongue into the gasket a distance sufficient to produce a tight seal against a lock pressure of 2 psig internal or external. Since a higher pressure in the vessel will produce a much larger gasket seating load than that required to seal the door, it is necessary to provide stops on the door frame to limit the

penetration of the door into the gasket under high temperature.

The locks will be equipped with electric lights and will have telephone systems to facilitate conversation between persons in the lock and persons inside or outside of the containment vessel.

At times when the power plant is not in operation, it may be desirable to have both doors in the open position so that personnel and large equipment can be transported through the lock with comparative ease and in a shorter time. In each lock, the interlock can be violated. This will make it possible to have both doors open simultaneously. Special tools and procedures will be required, so that the interlock can be violated only by specific authorization of the plant management.

A floor plate is installed in the personnel lock. The live load used in the design of the floor and lock is one hundred pounds per square foot. The spherical or multisphere construction principal used in this lock has economic and design advantages over a cylindrical lock. This is because the lock is located in the shell at a considerable distance below the equator of the sphere. The included angle between the horizontal axis of the lock and a radial line drawn through the shell at the lock is approximately 24°. The intersection of a spherical lock with the spherical vessel is a circle. The circular cutout with its reinforcing detail provides a scheme which can be fabricated more accurately and which provides a more uniform stress distribution than would be the case for an elliptical cutout required for a cylinder of the same diameter.

The floor in the equipment lock is designed to support truck wheel loads of ten tons per axle. Truck axles are ten feet apart. The outside dimensions of the door are greater than the frame in the bulkheads. Removable floor sections are provided in the area immediately in front of the exterior door. Since the truck will pass through the lock at infrequent intervals, the removable floor sections can be stored and installed only when necessary. There is a sub-floor under the removable sections which makes it possible to use the lock and operate the exterior door in the normal manner except when transferring the truck through the lock.

It is extremely unlikely that, under any operating conditions, the truck will be in the equipment lock coincident with the application of design pressures. For this reason, the net stresses which result in a combination of stresses produced by the heavy truck loads with stresses produced by design pressures, are not required to be within the Code allowable stresses.

Inspection and Testing

When it was evident that the ASME Boiler and Pressure Vessel Code had passed Code Cases which permitted the building and stamping of this type of vessel to Code requirements, insurance inspectors were hired to follow the work both in the shop and in the field.

All of the hand welds are inspected at least four different times. The butt welds are inspected using magnetic particle inspection after arc-air gouging the back side of the first side welded. The completed butt weld is fully radiographed and then it is inspected with soapsuds both before and after the overload test. All of the corner and fillet welds which cannot be radiographed are inspected using the magnetic particle method after, and in some cases during, welding. These welds are also tested using soapsuds both before and

after the overload test, and in addition, are inspected again using the magnetic particle method after the overload test.

Because the vessel is not capable of supporting a hydrostatic test, the standard pneumatic test specified by the Code of 1-1/4 times the working pressure will be applied to the vessel. After the vessel has been erected on the 20 tubular columns, the lower portion will be held at least 2'-6" above the rock excavation. The first pneumatic test will be made while the vessel is supported in this manner so that the entire shell can be inspected.

All welds and seals will be inspected for leaks using a soap bubble test while the sphere is under a pneumatic pressure of 5 psig. The test will be started with the inner door of each of the locks closed and the outer doors open. After inspecting the inner bulkheads and doors, the outer doors will be closed. Then the locks will be pressurized and the outer bulkheads, doors and the lock shell will be inspected for leaks using a soap bubble test.

If no leaks are found the pressure will be raised to 37 psig and then lowered to the design pressure of 29.5 psig. The soap bubble inspection described above will be repeated. During this inspection any increase in pressure due to an atmospheric increase in temperature will be bled off so that the design pressure is not exceeded. Because of the potential risk involved, no one will be allowed on or near the sphere during the overload test.

With the pressure at approximately the design pressure a leakage rate test will be conducted over a 48 hour period. After this holding time, the air will be released and the welded openings will be inspected both inside and out using magnetic particle inspection.

After cutting the temporary openings, no work will be done on the spherical pressure vessel until the internal equipment has been installed. When this work is completed the temporary openings will be rewelded and these seams will be inspected in exactly the same manner as were the original shell seams. A second pneumatic and leak rate test will be conducted; however, the inspection will be limited to the new welds since all of the other work will have successfully passed all previous inspections and tests.

During operation of the Dresden Nuclear Power Station shown in Figure 14, the loads on the structure will normally be small. However, the column load measuring devices will be available to provide a check on the uniformity of the column loads and the elevation of the equator in the remote possibility of unequal settlement. The locks will be tested to 2 psig and inspected using soapsuds in the shop. This tightness test may be used periodically to check the lock door seals and bulkhead connections without necessitating the pressurization of the entire vessel.

REFERENCES

1. ASME Paper No. 56-A-169 entitled "The Dresden Nuclear Power Station" by Messrs. Wolcott, Elliott, Peabody, Roy, Shanz and Sege.

2. Companion paper entitled "Civil Engineering Aspects of the Dresden Nuclear Power Station" by J. E. Love, C. S. Darrow and B. H. Randolph presented at February 1958 Meeting of ASCE.

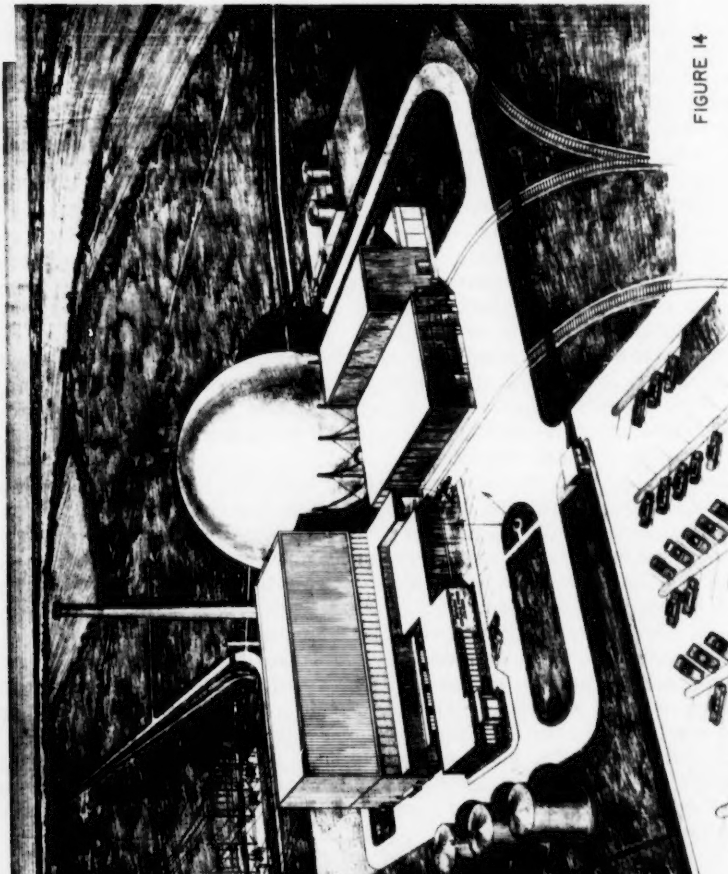

FIGURE 14

Paper 1602 April, 1958 PO 2

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

CIVIL ENGINEERING ASPECTS OF THE FERMI ATOMIC POWER STATION

Pharo C. Burg, A.M. ASCE[1] and John G. Feldes[2]
(Proc. Paper 1602)

INTRODUCTION

The Enrico Fermi Atomic Power Station under construction near Monroe, Michigan, will convert atomic energy into electricity through the use of a liquid metal cooled fast neutron breeder reactor. The fast breeder type of reactor differs from most other reactors in that it utilizes fast, high energy neutrons without the use of a moderator and it produces more fissionable material than it consumes.

The fuel for the reactor will be uranium molybdenum alloy enriched to approximately 25 per cent with uranium-235. The fuel material is in the form of zirconium clad cylindrical pins 30" long. These are assembled to form the core, approximating the shape of a right cylinder about 30" in diameter. Around this core is a blanket made up of unalloyed depleted uranium rods clad with stainless steel. Some of the neutrons which escape from the core during the fission reaction are absorbed by the uranium atoms in the blanket, which are subsequently transmuted into plutonium-239, a fissionable material. In this way new fuel is produced.

The heat produced during the fission process is carried out of the reactor to an intermediate heat exchanger by means of liquid sodium. In the intermediate heat exchanger, the heat is transferred to a secondary sodium system which then carries it to a once-through type boiler, where steam is produced to drive the conventional turbine. The intermediate sodium system is provided so that the primary radioactive sodium cannot contact water in the event of boiler leakage or failure. The use of a liquid metal coolant permits high temperature operation without requiring high coolant pressure. This is especially desirable in a radioactive system in order to minimize leakage.

The reactor is designed for an initial heat output of 300 mw and it is ultimately expected that this will be raised to 430 mw. The total sodium coolant

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2. Health Physicist, Power Reactor Development Co., Detroit, Mich.

flow is 132,000,000 lb/hr, resulting in an average coolant temperature rise across the reactor of 250 F at the 300 mw level. The sodium coolant temperatures for this case are 550 F at the inlet and 800 F at the outlet. At this level of operation, the reactor plant will produce 1,015,000 lb of steam at 600 psig and 650 F.

Throughout the design, safety has been a primary consideration. The low pressure of the coolant systems, the intermediate sodium cycle, and the gas-tight steel reactor building are a few examples of plant safety features.

The reactor plant construction cost estimate is $44,200,000 which includes $9,000,000 for research and development, plus allowances for working capital and administrative expense. Since this estimate is now about one year old, it is currently being reviewed and it appears likely that the revised estimate will show a cost increase in the order of ten per cent.

Site Development

The Enrico Fermi reactor plant is being built near the center of a tract of land of approximately 915 acres owned by the Detroit Edison Company. It is located about thirty miles from Detroit on the western shore of Lake Erie at Lagoona Beach, Frenchtown Township, Monroe County, Michigan. Figure 1 is a location map of the site. The City of Monroe with a population of approximately 20,000 is 7-1/2 miles to the southwest.

The property is generally low and marshy. During the summer of 1956, 100,000 cubic yards of muck were excavated and were replaced with 500,000 cubic yards of clay fill. This located the grade elevation about ten feet above normal water and was necessary to provide a suitable area on which to carry out the construction work. About eleven acres were thereby made available. The completed plant consisting of all the nuclear buildings plus the turbine-generator building will cover about four acres.

Because the plant will, during normal operations, produce large amounts of fission products in the fuel and will develop radioactive wastes which must be processed for disposal, some rather unique investigations were undertaken in regard to the site.

For example the protection of the site and structures from lake effects is of special importance at the Fermi Plant. This is to insure that the lake does not cause flooding in specific plant areas such as the holdup basins shown in Figure 2, and thereby release any radioactive effluent which might be contained in the basins. A study of maximum and minimum elevations under a combination of the most extreme conditions of lake level, and wind tide, plus maximum wave heights, has been completed. Preliminary engineering studies have been made of a sea wall, dike and breakwater for protection of the site. Further investigations are to be undertaken before the final design for shore protection is fixed.

Another special problem caused by the presence of radioactivity is the treatment of runoff from the portion of the site on which the reactor facilities are located. In the event that a spill of radioactive material should coincide with a period of rainfall, it would be imperative to retain control of the runoff which might be contaminated in excess of the maximum permissible concentration. To accomplish this, all storm drains which collect runoff from areas where radioactivity is present, will discharge to the

semicircular holdup basins (Figure 2). In this way the effluents can be monitored and, if necessary, held for decay and treatment before discharge.

Reactor Building

The integrity of all components is essential to the safety of personnel and equipment. The hazards due to unequal settlement must be eliminated. Therefore all structures either rest directly on rock or are built on piles that extend to rock which occurs about thirty feet below grade.

Since the major components of the liquid sodium system are to be tested before the reactor goes critical, the reactor building is the first structure to be constructed (Figure 3). The building is to house the reactor, sodium pumps, intermediate heat exchangers, piping, and equipment decay tanks. It is designed to be airtight and to provide for the following conditions: (1) The containment of the chemical products of a sodium-air reaction caused by a major type rupture or a large instantaneous energy release. (2) The containment of shock waves and missiles that might result from an instantaneous energy release.

The containment vessel is a 72 foot diameter cylindrical unfired pressure vessel with a hemispherical top and hemiellipsoidal bottom. Its over-all height is 120 feet and its center portion is a straight cylindrical section 66 feet high. The shell was designed in accordance with Section VIII of the ASME Boiler and Pressure Vessel Code. All welds were radiographed. The shell is fabricated from ASTM-201 firebox quality Grade B ordered to requirements of ASTM Designation A-300 plates using 1.03 inch plates in the cylinder and bottom, .52 inch in the dome and 1.25 inch at the knuckle line. No field stress relieving was done on the vessel shell.

The layout of the interior of the building places the operating floor 51 feet above the bottom of the containment vessel. The portion below the operating floor outside of the steel shell requires a biological shield wall seven feet in thickness. The first studies placed the foundation of the building on top of the rock at Elevation 554. This would put the operating floor about 28 feet above grade. Since all the doors are located above the operating floor, an elevator, or ramp, or both, would be required for personnel and equipment to enter or leave the building. Consequently the entire building was lowered twenty feet. This put the bottom of the foundation into the rock and located the operating floor only eight feet above grade. Access to the operating floor is by means of a 7 foot fill around the reactor building.

Another advantage of lowering the building farther into the ground is the substitution of earth shielding for some concrete in the biological shield wall. By this arrangement the lower section of the wall was reduced to three foot thickness.

A minimum clearance of 4' -6" extending around the vessel and 3 feet beneath the vessel was requested in order to make the welds and examine them during an internal pressure test of 40 psi. To obtain the clearance between the foundation and the underside of the vessel, temporary steel columns were used both to support and anchor it during construction.

After test, a concrete fill was placed between the steel vessel and the concrete foundation to cradle the bottom. Concrete was placed inside to anchor the vessel and to make a floor and a base for the reactor. Because of the inaccessibility of the space beneath the vessel and the fact that there was to

be a slight head of concrete on the bottom of the vessel it was decided that most satisfactory results could be obtained by placing dry aggregate and pressure grouting beneath the vessel. Layers were alternately poured outside and inside the vessel in order not to overstress the bottom plates during construction. The entire lower floor area and concrete equipment bases are covered with carbon steel and the joints are welded continuously to prevent any possible contact of solid, liquid, or gaseous sodium with the concrete.

The secondary shield wall surrounding the reactor vessel (Figure 4) has two functions; first as a shield to absorb neutrons and gamma rays; second as a barrier in case of an instantaneous energy release. Concrete of 144 lb/cu ft density and carbon steel make up the materials for the wall. Thickness of the wall was determined by the shielding requirements and varies from 31 inches to 40 inches. The shape of the wall is dictated by the equipment and piping in its vicinity.

The destructive effects of burning sodium requires that the concrete be protected from contact with sodium and sodium vapor. One-half inch welded steel plates completely enclose the wall and are welded to the floor plate both at the top and bottom of the wall.

The space around the wall is limited and these plates will serve as formwork. Sections A and B show how the plates on the two sides of the wall are fastened together with adjustable rods which will act as ties and spacers during construction and anchors after completion.

In certain areas on the outside of the wall gamma ray shielding is required in the form of carbon steel up to two and one-half inches in thickness. A space is provided between these plates and the wall for the circulation of inert gas in order to maintain a given temperature in the wall. A system was devised by which these plates would serve the shielding requirements and at the same time be small enough to be erected after piping is placed and requires a minimum of field work and welding (Figure 4, Section B-B). Plates 10 x 1/2 with two angles attached are to be welded to the wall plate. As the shielding plates are stacked on edge vertically, short angles will be welded to the 10 x 1/2 plates to hold the shield plates in place.

The consideration of the wall as a missile barrier requires the wall to span as a slab between floors. Within the concrete wall reinforcing steel bars are placed in two layers. In each layer No. 7 bars are spaced 10 inches on center horizontally and 12 inches on center vertically. The steel columns placed within the wall tend to reinforce it in case of a horizontal load.

From each of the three sodium units within this building several pipes penetrate the secondary shield wall. The pipes vary in size from 2 inch diameter to 30 inch diameter and enter the wall at various angles. Each pipe receives special sleeve consideration. Erection of the component parts of the wall demands rigid adherence to the planned sequence.

The operating floor serves also as a radiation shield and a support from which equipment is suspended. As a shield the thickness was established to be five feet. A layer of steel plates several inches thick form the lower portion of the floor. Above this is the concrete as is shown in Figure 3. Structural beams are supported on columns as is indicated in Figure 4. The irregular pattern is necessary in order to clear the equipment which extends through and below the floor. The beams are designed only to carry the steel plates and the wet concrete plus pipe loads. The thick reinforced concrete slab is designed to carry its own dead load, equipment loads suspended from it, and certain live loads.

Fuel elements will enter the building in a cask car weighing about 150 tons. Track girders will carry this load to the columns.

Provisions have been made in the design to accommodate a special railroad car on the operating floor for the removal of equipment to be repaired. It is expected that this will occur infrequently.

Sodium Building

The second building to be constructed is to house the sodium storage tanks and process equipment (Figure 5). Because radioactive sodium is processed, the building walls are required to perform the duties of shielding and the walls and roof of the tank room are 2' -6" thick. The walls and roof of the cold trap room are 6 feet thick and are lined with carbon steel plate 1/4 inch thick. There are no conventional doors or windows in these rooms. Until the plant goes critical, access will be through openings left in the wall.

When operation is about to begin these ports will be closed by concrete blocks so made that the joints will be broken in two directions. This is one of the interesting angles to the shielding field: all expansion joints and construction joints must be stepped. Streaming of gamma rays may result if a plane surface extends through a structure to be used for shielding.

To eliminate the necessity of forms and shoring for the roof, a system was worked out to use precast concrete slabs. These will serve first to support the wet concrete until it has set, and then they become an integral part of the roof. Likewise, the steel plate lining will double as inside forms for pouring the walls of the cold trap room.

Pipe piles 12 inches in diameter which carry loads of 50 tons support the building. Reinforcing steel is placed in both faces of all walls and slabs.

Sodium Tunnel

The sodium tunnel, a section of which is shown in Figure 6, houses four pipes four inches in diameter to carry sodium between the reactor building and the sodium building. Here the radioactivity of the sodium needs a shield of concrete 6' -6" thick.

The temperature of the air inside the tunnel is expected to reach 150 F. Expansion joints separate the five sections of the L-shaped tunnel which is approximately 90 feet long. Copper water stops are placed in all joints of the concrete. Construction joints and expansion joints are stepped to prevent streaming of gamma rays and a shield bar of steel is installed in the step. Glass fiber material fills the space in the joint.

This structure is lined with 1/4 inch carbon steel plates welded airtight to prevent any possible contact of sodium with concrete or water. Each section of the tunnel is supported independently on 12 inch diameter pipe piles. Because of the limited working space within the tunnel, the sequence of erection will be as follows:

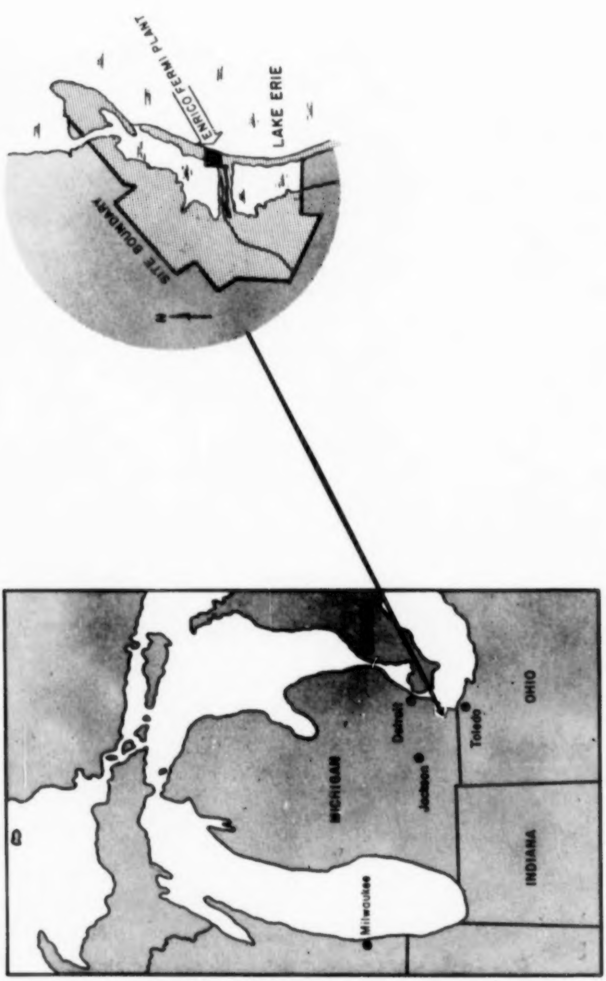
1. Construct the bottom and sides including the liner plates.
2. Install the four sodium pipes complete with their secondary containment, insulation and heating coils.

3. Place and seal weld the plate liner cover.

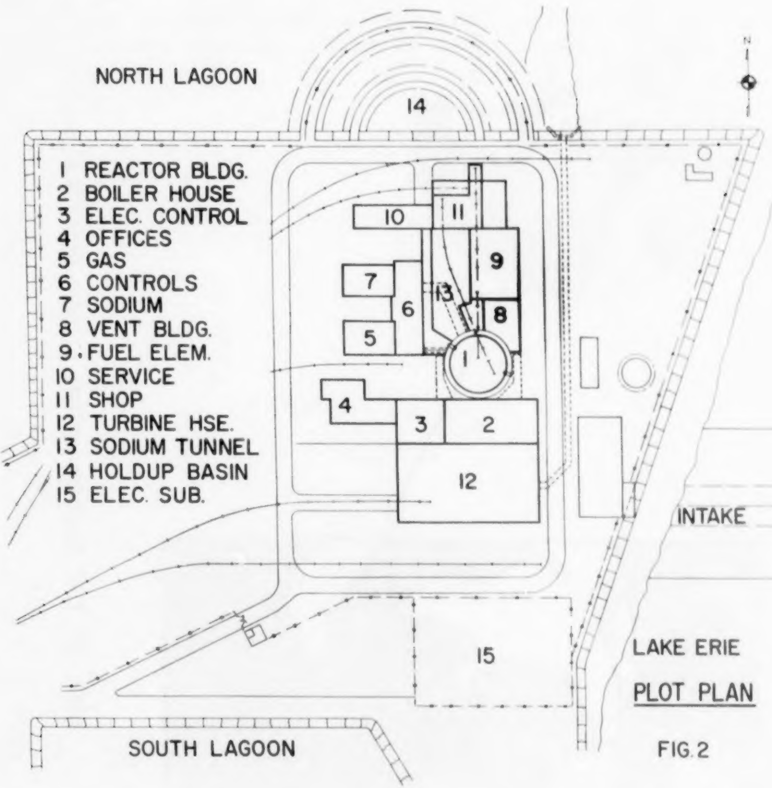
4. Pour the concrete shield top.

Constituent Buildings

Another look at this project can be taken as plans for component buildings are completed. A cutaway isometric (Figure 7) shows the reactor building in the foreground with the proposed boiler and turbine houses to the right.

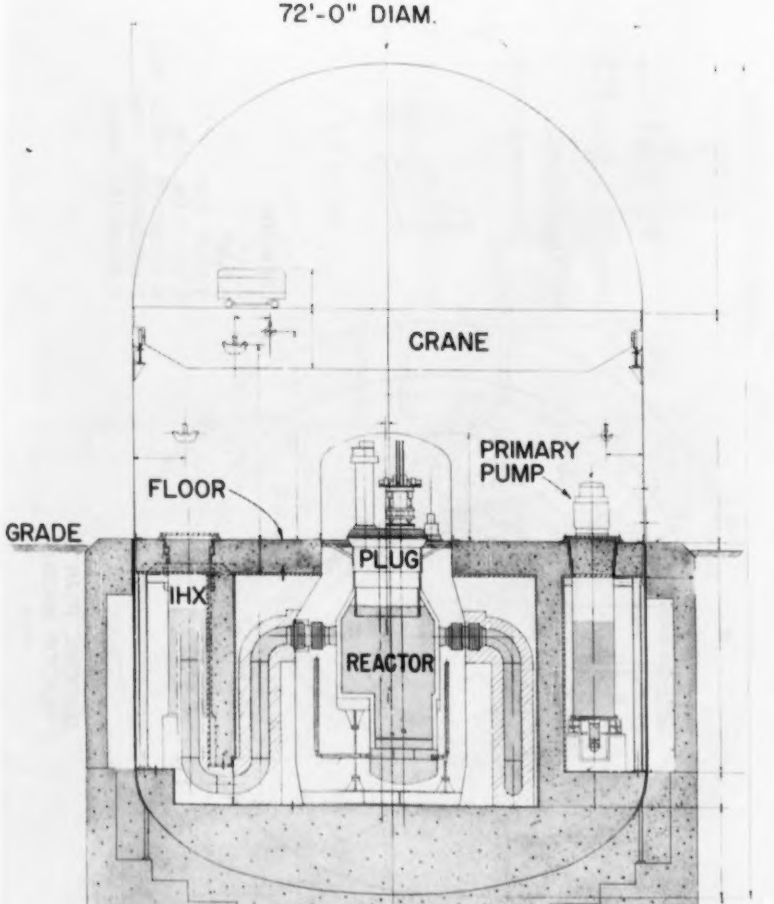


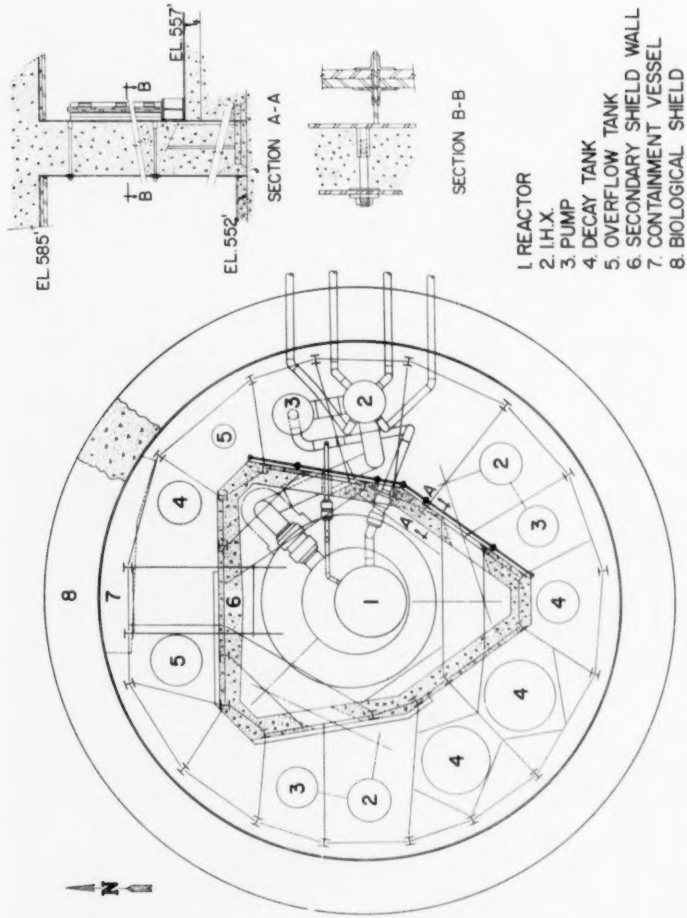
SITE OF THE ENRICO FERMI ATOMIC POWER PLANT



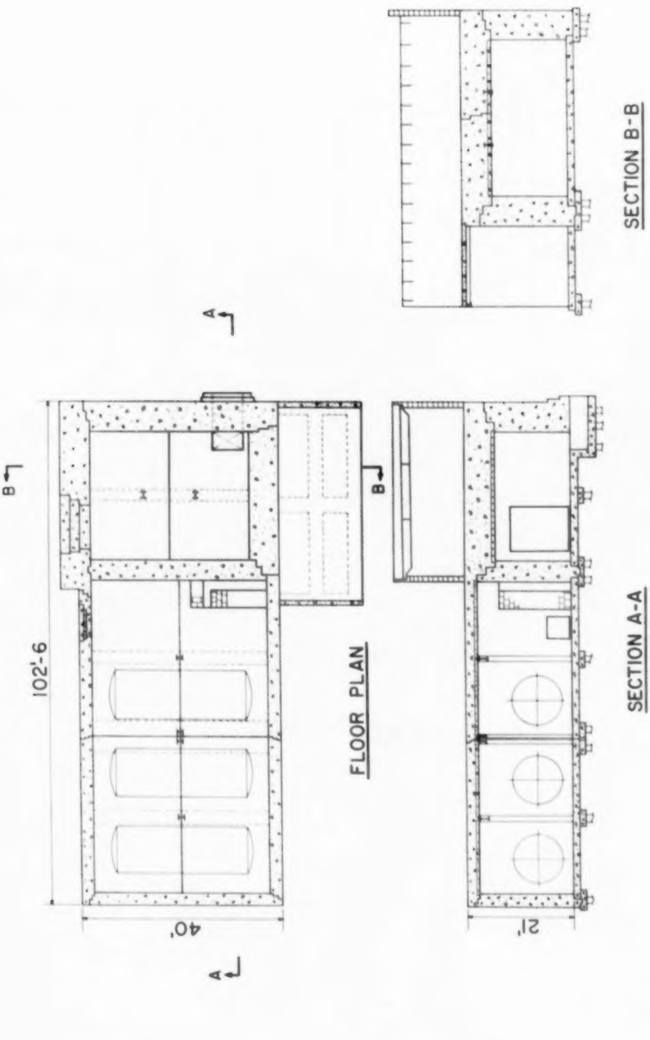
PLOT PLAN

FIG. 2

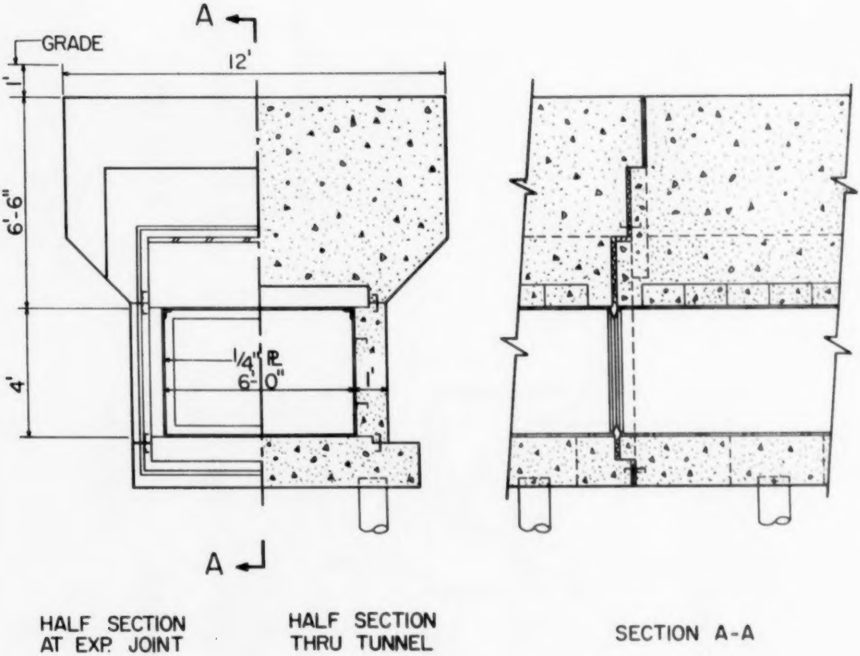

72'-0" DIAM.
CRANE
PRIMARY
PUMP
FLOOR
GRADE
PLUG
IHX
REACTOR



SECTIONAL PLAN OF REACTOR BUILDING
FIG. 4

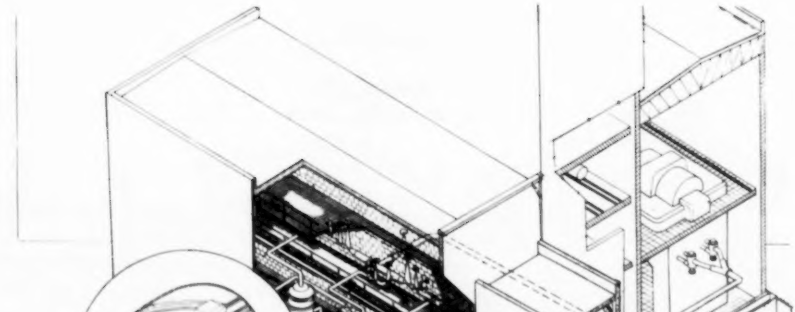


SODIUM BLDG.
FIG.5



SODIUM TUNNEL

FIG. 6



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Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1618)

PENSTOCK EXPERIENCE AND DESIGN PRACTICE[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—Mr. Richards is to be congratulated on this paper which will be of material value to the profession. The design, fabrication, construction and operating experience on the group of penstocks designed and built by Mr. Richards' company since 1948 is indeed almost unique in its range of static heads and diameters.

The consistent use of ASTM A212B steel, with A285B used where thinner plate meets requirements, confirms in very large part other North American experience. However, it is to be observed A242 steel was employed for the shaft section of the Haas penstock where the field joints are x-rayed and field stress-relieved at the extreme lower end. This appears to represent the first substantial use by Mr. Richards' organization of welded field joints.

It would appear that many of the developments reported in the paper are located at altitudes where they are subjected to rather extreme temperature differences. The value of the paper would be enhanced if the author discussed this point with respect to steps taken to ensure freedom from brittle fracture of the steel. For instance, are any tests for brittle fracture made on coupons from the plates used in fabrication of the penstock?

The value of the paper would also be enhanced if the author noted field procedure to correct damage during transportation and field handling, as it is generally impossible to prevent some scoring of penstock sections at one time or another.

a. Proc. Paper 1397, October, 1957, by Gordon V. Richards.

1. Chief. Engr., Power Dept., Aluminum Labs. Ltd., Montreal, Canada.

LARGE SPIRAL CASINGS OF T-1 STEEL[a]

Discussion by F. L. Lawton

F. L. LAWTON,[1] M. ASCE.—The paper is of very considerable interest as a step forward in the use of thinner and lower-weight spiral casings for Francis turbines. The author indicates the use of T-1 steel results in a reduction from 1-5/8" maximum thickness required with ASTM 285 Grade B mild steel to 1" with Tl for the Noxon Rapids Francis turbines. The cost of field welding is stated to run to about twice the cost of shop welding for ASTM 285 steel and 2-1/2 times for Tl steel where shop welding of Tl costs 1-1/2 times that for the same thickness of ASTM 285. Have these indicated relative costs been substantiated by field experience?

Reference is made to the use of Tl steel being advantageous because of its excellent property of notched toughness. Is this appraisal based on its behaviour during welding operations or service conditions?

It may be noted the Shipshaw (Canada) 100,000 hp, 128.6 rpm, vertical Francis turbines have completely welded spiral casings. The inlet diameter of the casing is 16 ft. and the maximum out-to-out diameter 49 ft. 8 in. Placed in service in 1942-3, these units have given entirely trouble-free performance.

a. Proc. Paper 1398, October, 1957, by E. L. Seeland.
1. Chief Engr., Aluminum Labs. Ltd., Montreal, Canada.

COEXISTENCE OF FISH AND DAMS[a]

Discussion by Mons H. Benson

M. H. BENSON,[1] M. ASCE.—It is gratifying to note the feeling of optimism the authors have that there can be coexistence of fish and dams, and their conclusion that where apathy was once quite evident when the subject of adequate care for migrating fish was considered, careful study and desire to protect the fishery are now evident. This has come about by an increasing general interest in the preservation of wildlife resources, pioneered by a comparatively few persons and organizations whose devotion to this cause has transcended all obstacles in their belief that progress does not necessarily mean destruction of our wildlife heritage. These people are not impressed by arguments concerning cost but believe the cost of mitigating harmful results should be included as part of the whole study of the development, and this feature should have equal rank in importance with any other feature of the proposed works. The feeling of the wildlifers, to use a term which seems to be coming into general use, goes beyond a mere desire to partake of a larger harvest of natures bounty, but stems from a love of the outdoors and a desire to preserve and to leave these natural resources in better shape after they are gone. They know our descendants will thank their forebearers not for how much was saved in necessary improvements, but for the natural resources that were bequeathed to them along with these improvements.

Messrs. Preston and Rydell have made a valiant attempt to show the progress made to date in solving the problem of anadromous fish migration past dams. It would have been nice if they could have presented a solution for each type of case, but this lies in the future. Much work remains to be done to find a satisfactory answer for each of the many different situations at different dams. However, the progress outlined to date by the authors contains much for which we can be grateful. The experiments made at Elwha and Glines Canyon dams give results that are very promising, particularly those in connection with the passage of yearling silver salmon through a Francis type turbine. A survival rate of 69.6 percent through this type of turbine under a head of 184 feet is truly remarkable, and would maintain the runs while additional research is being carried on to develop improved methods of fish passage for the downstream migrants. The results obtained at the 230 foot high Baker dam in the passage of fingerlings over a ski-jump spillway are also remarkable. With a survival rate of 85 percent this type of construction, where feasible, holds great promise. Also promising is the skimming device with the attraction water created by low head pumps. All of these methods give assurance that the fish runds do not have to be destroyed.

a. Proc. Paper 1414, October, 1957, by Howard A. Preston and Louis E. Rydell.

1. U. S. Bureau of Reclamation, Boise, Idaho.

In reporting on all the methods of downstream passage of fingerlings over high dams the authors were limited to the results obtained by experiments to date, and many improvements and refinements will undoubtedly be made. What these improvements and refinements will be must lie in the field of speculation, and even the progress made so far must be speculative until proven by the fish runs themselves in future years. Until such time as tested methods are assured, the immediate problem is one of maintaining the present runs. This appears possible from the tests reported by the authors. Particularly intriguing is the possibility of improving the safe passage of fingerlings through Francis turbines along with the use of other methods. It may be that the biological structure of fish is such that they can withstand far greater positive pressures than has been realized, and as some of the test results seem to indicate. At least the positive pressure builds up during the period of time that the fish passes down the penstock. However, as the fish passes through areas of negative pressure such as an area of cavitation within the turbine, or as it passes out the turbine into the draft tube, the expansion of nitrogen and other gases within the fish is immediate. With a head of 184 feet and a turbine setting 25 feet above tailwater, this expansion could be twenty times. It would be of interest to know if the two tests reported on the passage of fish through Francis turbines were made during a period of high tailwater, thereby reducing the negative pressure in the draft tube. If so, this would open up an interesting field of speculation that damage to fish could be reduced by avoiding high negative pressures in the draft tube. Reducing these negative pressures would require that the turbine setting be close to the tailwater elevation.

The upstream passage of adult salmon over high dams needs little comment as the authors point out the various ways in which this has been done. Even during temporary construction conditions at McNary Dam they report as many as 15,000 salmon were passed over the dam in one day. The important point in this connection is that provisions for upstream passage of anadromous fish must be made or there is no hope of maintaining the run. Many of the runs which have been destroyed in the past have been the result primarily of the adult fish not getting past the dams on their upstream migration, and not as a result of the fingerlings being unable to survive the downstream passage.

Messrs. Preston and Rydell mention their hesitancy in undertaking the preparation of this paper, being engineers and not biologists. This is quite understandable and it is to be hoped the biologists will be able to provide some of the answers which are so badly needed. In the meantime the authors are to be commended for recognizing this problem with a society paper on the subject. In our growing economy water resource development will continue, but the conservation of wildlife resources is also important, and should rank in its own right as a partner in this development.

FIXED-WHEEL GATES FOR PENSTOCK INTAKES[a]

Discussion by Joseph R. Bowman

JOSEPH R. BOWMAN,[1] A.M. ASCE.—The author is to be complimented on a most comprehensive coverage of the design of high-head, fixed-wheel gates. His paper is the culmination of many years of research and operational experience, on the part of the engineering staff of the Bureau of Reclamation, in the development of this type of gate for deep conduit intakes. The Bureau has shown conclusively that the fixed-wheel gate is well suited to a large portion of the field formerly dominated by the caterpillar gate. (The term "caterpillar," as used herein, is intended to include tractor, coaster and other continuous roller-train gate sub-types.

The relative merits of fixed-wheel and caterpillar gates are concisely summarized. However, one important advantage of the former type has not been mentioned; and that is that the deflection of the gate leaf is not as critical as in the latter type. Deflection produces end rotation which, in turn, tilts the load-bearing assemblies of the gate. In the case of a fixed-wheel gate, a wheel bearing on a crowned track can be tilted by an appreciable amount without materially changing its loading characteristics; hence a relatively large amount of gate leaf deflection can be tolerated. On the other hand, tilting the roller assemblies of a caterpillar gate results in loading the rollers eccentrically, inasmuch as they are cylinders bearing on flat track plates. In order to limit the loads at the heavily loaded ends of the rollers to safe values, it usually is necessary to limit the deflection of the gate leaf by reducing its working stresses. This characteristic tends to limit the caterpillar gate to lesser widths than the fixed-wheel gate at certain heads. A number of schemes have been devised to compensate for eccentric roller loading without penalizing gate leaf design, but to date they leave much to be desired.

The caterpillar gate is not nearly as "frictionless" as some laboratory tests might indicate. Quite a few gates of this type have failed to operate properly as the result of the assumption of too low a coefficient of friction. Values indicated by tests on single rollers or on models are almost impossible to attain in prototype. The clearances and tolerances among the roller-train components, which are necessary to assure free movement, tend to throw many of the rollers slightly askew; their consequent "crabbing" motion introduces some sliding friction in addition to rolling friction. Thus the overall coefficient of "rolling" friction may be as great as two or three times the values ascribed to pure rolling motion. The writer notes that both he and the author agree on a "rolling" friction coefficient of 0.10.

a. Proc. Paper 1420, October, 1957, by Sylvan J. Skinner.

1. Civ. Engr., Erik Floor and Associates, Inc.; Chicago, Ill.

The tabulation of conduit entrance proportions shown in Fig. I is a welcome addition to the gate designer's store of preliminary short-cuts. How often one has wished that it were not necessary to design the penstock entrance in order to obtain preliminary gate dimensions.

The writer is in full accord with the author in that decisions as to the intended function and operation of a gate should be reached well before the selection of gate type is finalized. Moreover, the gate and its operating equipment should be treated as an integral unit; if the gate and hoist are considered independently, an incompatible combination may result.

Estimating Data

The parameters used in the weight curves of Fig. IV differ widely from those which have been developed for fixed-wheel gates at lower heads.(1) At the higher heads the hydrostatic pressure is distributed nearly uniformly over the gate, and the horizontal girders can be uniformly spaced. For this reason, the weight parameters need not be dependent on the gate height. At lower heads, on the other hand, the hydrostatic pressure varies considerably over the height of the gate, and the spacing of the horizontal girders is varied so that they assume equal shares of hydrostatic load. As a result, the weight of the gate is not uniform throughout its height, and the weight parameters necessarily include the gate height. The writer has found that the ratio of head on gate sill to gate height is a useful parameter for marking the distinction between high-head and low-head submerged gates. When the ratio does not exceed six (6), low-head weight parameters and design procedures should be utilized; conversely, if the ratio exceeds seven (7), high-head criteria should govern. A parameter of this nature will, of course, result in a few cases in which there will be no clear line of demarcation, and wherein the final classification will be resolved as a matter of preference; however, this head-to-height ratio can have considerable value in preparing quick preliminary estimates.

Design Details

The writer observes that the Bureau now uses the self-lubricating "Lubrite" type of wheel bushing exclusively on its fixed-wheel gates. In most high-head applications, this type is superior to grease-lubricated, anti-friction bearings. Prevailing water temperatures at deep intakes are so low that the grease in the housing of an anti-friction bearing tends to harden, producing locked-wheel conditions. Furthermore, a bearing seal that must perform the dual function of retaining the grease while keeping out water under high heads, and which must do so without introducing excessive friction on the axle against which it seals, is scarcely a run-of-the-mill item.

In Fig. VII it is noticed that the Bureau gates apparently make use of riveted connections to the exclusion of welding. Field assembly by riveting is often preferred to field welding because the distortion that may arise from welding is eliminated. However, in view of the advanced welding techniques currently available, the writer fails to understand why shop welded fabrication has not been utilized more extensively by the Bureau. It is further observed that skin plates are riveted to their supporting frameworks; a good many gate designers, the writer included, have felt that this practice became obsolete as more reliable welding techniques were developed. Moreover, it

has long been recognized that rivet heads are focal points for corrosion of a skin plate.

Standardized Designs

In contrast to the relationship between the private engineer and his client, the engineering staff of the Bureau of Reclamation is in the enviable position of being its own client. As such, the Bureau staff can plan its requirements well into the future. This situation promotes a high degree of standardization in design criteria and procedures. It also encourages the development of standard type designs that can be re-used many times with relatively little modification; the prospect of frequent re-use of a basic design economically justifies refinements in design procedure that many private engineers would not consider worthwhile. In his dealings with a number of different clients, the private engineer, as a rule, must tailor his designs to conform to widely divergent client preferences as to design criteria, operational requirements and numerous other governing factors. Consequently, opportunities for development of refined standardized designs are few and far between. By and large, however, gate designers in private practice have been inspired to many noteworthy achievements by the Bureau's bold and successful venture into the field of high-head, fixed-wheel gates.

REFERENCE

1. Boissonnault, F. L., "Estimating Data for Reservoir Gates," Transactions ASCE, Vol. 113, p. 992, 1948.

CIRCULATING WATER SYSTEMS OF STEAM POWER PLANTS[a]

Discussion by Clifton W. Bolieau

CLIFTON W. BOLIEAU,[1] M. ASCE.—The author is to be congratulated for his comprehensive description of the circulating water systems for modern steam power plants. During the past ten years the Tennessee Valley Authority has built seven such plants, including the world's largest, and has successfully solved the many problems involved in these systems. In general, the design of each system is similar to that used on the Johnsonville Steam Plant described in the writer's article in Civil Engineering, June 1953, and will not be repeated here. One major change in subsequent plants has been the substitution of full-size hydraulic cylinder controlled butterfly valves for similarly controlled reduced-size cone valves on each circulating pump discharge. This resulted in entirely satisfactory operation at about the same head loss and with a considerable savings in first cost. The Johnsonville article described the 78-inch precast concrete pipe used for each unit. This type of conduit, except for some increase in size, was also used at the Shawnee, Widows Creek, and Kingston Steam Plants. At these three plants rock for tunnels or foundation was either impractical or nonexistent. To save costs these pipes are laid with the least practical cover which, coupled with a large variation in lake levels, results in pump discharge conduits operating with hydraulic gradients below the pipe during minimum lake levels. Vacuum systems were provided in the pumping stations at these plants for use whenever this flow condition exists. Visitors to these plants often question the accuracy of the pressure-vacuum gauges on the discharges of the circulating water pumps when they are operating at negative pressures. At TVA's three other plants, namely Colbert, John Sevier, and Gallatin, concrete lined rock tunnels were used as circulating water conduits. Depth was desirable and pump discharge connections were made as low as practical to simplify the transitions to the tunnels. At these plants all conduits and pump discharges are well below the hydraulic gradients and no vacuum systems are provided for their use.

At all the TVA plants velocities throughout the circulating water systems, except at pump discharge and valves, have been maintained at about 7 feet per second. The number of bends in the conduits have been reduced to a minimum, all are of long radius, and transitions are designed to cause the minimum hydraulic disturbance. For all stations except one this results in total dynamic heads of from 20 to 22 feet, of which about 11 feet is the loss through the condenser tubes and water boxes. The one exception is TVA's Shawnee Steam Plant located on the Ohio River. Variations in river levels of up to 57 feet are anticipated. To cover this range without exceeding

a. Proc. Paper 1488, December, 1957, by R. T. Richards.

1. Prin. Mech. Engr., Div. of Design, Tennessee Valley Authority, Knoxville, Tenn.

practical negative head conditions in the discharge side of the condenser, a weir was placed in the discharge canal to introduce about 12 feet of static head into the system at low river level. Minimum river stage occurs almost continuously during the summer months as controlled by a navigation dam a few miles downstream. Except at low levels the discharge butterfly valves on the condensers must be throttled proportionately to introduce artificial static head to compensate for that lost as weir height is reduced. This is necessary to prevent excessive flow through the condenser tubes which would cause cavitation.

At the Johnsonville Steam Plant the discharge of the circulating water back into the lake was through a diffusing section in an effort to reduce the total operating head. At subsequent plants the extra cost of this structure could not be justified for the small amount of head recovered. At all other plants the conduits end at a head wall and a full velocity head loss is included in the total dynamic head of the pumps.

The author's experience with over-designed pumps is interesting. With one exception, the TVA experience is similar. However, at only one plant has difficulty with submergence been obvious. Here, at minimum water levels and with 10 feet of pump suction submergence, vortices formed at the surface down through which air was drawn into the pump suction. This resulted in noisy, inefficient operation which was stopped temporarily by hanging baffles about 2 feet below the surface ahead of each pump. Some small but not air carrying vortices still formed. Better results were obtained by extending the suction bells of the pumps horizontally which greatly reduced entrance velocities and eliminated entrance of air. All pumps at this plant have been so modified and smooth operation at all water levels is obtained.

No vacuum breakers, air chambers, open manholes, or other devices have been provided in TVA's systems for emergency control of water hammer and no evidence of water hammer has been reported. Great care was taken in the design of the pump control system to prevent water hammer or surges. The condenser water boxes are structurally the weakest parts of these systems and could not possibly withstand the water hammer pressures mentioned by the author. In each of TVA's systems the test pressure of the water boxes is equal to or greater than the pressure which could be applied by the pumps at shutoff head at maximum lake level. This assumes the rather improbable condition of closure of the condenser discharge valves during unit operation. This design condition results in water boxes at one station with 40-psi test and 25- or 30-psi test at all others.

At all but one of TVA's 48 operating or authorized major steam units, two pumps discharge through a wye connection to a separate conduit for each unit. Each pump has a hydraulic cylinder-operated discharge valve controlled by a direct-current solenoid valve. The cone or butterfly valves are arranged to open slowly and close rapidly. One pump is started at a time and its motor and valve energized at the same time. The pump reaches shutoff pressure very quickly but flow through the system starts slowly because of the throttling action of the valve. When the valve is open and flow stabilized, the second pump is similarly started and flow slowly increases again as its valve opens. Upon stopping the system, one pump motor and its valve is deenergized. The valve closes rapidly to prevent backflow through the stopping pump. The flow in the system reduces slowly because the inertia of the long column of water actually increases the flow through the second pump. This

is helped by the increase in its capacity as the head is reduced. When the second pump is stopped its valve is locked open and again the inertia of the water in the system causes flow to continue at a decelerating rate until stoppage. The vacuum system in the powerhouse for venting accumulated air in the discharge water box of the condenser holds water up in the condenser preventing any separation.

The one exception to this description is the system now being designed for the seventh unit at the Widows Creek station. This will be the world's largest generating unit with a capacity of 500 megawatts. Three pumps will be provided discharging through a double wye into a single conduit. Two speed electric motor operators are being specified for the pump discharge butterfly valves instead of the formerly used hydraulic operators. Except for the number of pumps a similar starting and stopping sequence will be used.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347 (BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c], 1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4), 1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

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FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

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c. Discussion of several papers, grouped by divisions.